SMALL-DIAMETER PILE SEGMENT TEST
AT EMPIRE, LOUISIANA
Volume I

By

The Earth Technology Corporation
Houston, Texas

August 30, 1984

THIS REPORT IS CONFIDENTIAL

The contents of this report are proprietary in nature and shall remain confidential for a period of three (3) years following the date of issue of this Report.

TABLE OF CONTENTS

Volume I

TABLE OF CONTENTS (continued)

PREFACE

The work reported herein is a part of a continuing research effort at The Earth Technology Corporation which is directed toward increasing the understanding of the mechanics of the behavior of axially loaded piles, under the guidance and direction of Hudson Matlock, the Vice President for Research and Development at The Earth Technology Corporation.

The Earth Technology Corporation developed and proposed a program of research in which interested companies could participate by purchasing the results for a fixed fee. Several companies joined, including: Chevron Oil Field Research Company, Conoco, Inc., Gulf Exploration and Research Co., Mobil Research and Development Co., Sandia Corporation, Shell Development Co., and Union Oil Company.

The project team for The Earth Technology consisted of Dewaine Bogard, Wei-Chou Ping, Fleet Brown, Neil Dwyer, Scott Bamford and Jean Audibert. Dewaine Bogard planned the testing program, directed the field experiments, supervised the data processing and wrote the report. He was assisted in the field by two project engineers, W.-C. Ping and S. Bamford; and a senior technician, F. Brown. N. Dwyer provided assistance with the electronics when needed. J. Audibert served as project manager, reviewed the report and provided overall supervision.

Special notes of appreciation are due Messrs. George Sgouros of Shell Development Co. and John Bigham of Chevron. G. Sgouros graciously hosted the presentation of the proposal and, later, the presentation of the results. Mr. Sgouros also served as a third-party reviewer of an initial draft of the report; his comments and suggestions are gratefully acknowledged. J. Bigham offered valuable advice regarding the securing of permits for the site, access road and construction. He also furnished valuable insight into the early site history and previous testing programs.

We would also like to express our appreciation to those who visited the site during the course of the work: Messrs. John Bigham and John J. Rentschler of Chevron, U.S.A., Inc.; Louis Long of Mobil Research and Development; Bob Prindle of Sandia; and George Sgouros of Shell Development Co.

ABSTRACT

A series of experiments have been performed at the site of a large-scale pile load test at Empire, Louisiana, using two types of instrumented pile segment models. The pile segment models are 1.72 and 3.00 in. in diameter and were instrumented to measure the total radial pressure, the pore pressure, the shear transfer, and the relative pile-soil displacement.

Experiments were performed at six depths, ranging from 120 to 250 ft below the ground surface. Two sets of three depths each were selected, so that the experiments would be performed at depths corresponding to the top, middle and bottom of the embedded lengths of two of the 14-in. diameter piles. The pile segment models were driven and pushed into the soil. The larger-diameter model was used with both open-end and closed-end cutting shoes, in order to investigate the effects of plugging on the soil pressures, the rate of consolidation, and the shear transfer.

Load tests which were performed included: slow monotonic loading to failure in tension, one-way cyclic tension loading at a constant bias load but with varied levels of cyclic loads and two-way displacement-controlled loading, with failure in both tension and compression.

The results of the monotonic load tests performed during the experiments generally agreed well with the results of the earlier pile tests on 14-in. diameter piles, at the same degree of consolidation.

The effects of diameter and degree of cavity expansion (plugging) on soil pressures are clearly demonstrated by the results obtained during this field testing program. Using full-displacement models of different diameters, the consolidation times were confirmed to be approximately proportional to the squares of the diameters. When comparing the closed- and open-end models of the same diameter, the consolidation times were shown to vary with the degree of cavity expansion.

The effects of consolidation time on the limiting shear transfer were also demonstrated, with the magnitude of the limiting shear transfer being linearly

related to the percentage of dissipation of the excess pore pressures which were created during the installation of the probes. Due to the decrease in the radial total pressures with time after installation, the limiting shear exhibits a nonlinear relationship to the radial effective pressure; the degree of consistency in the results of the several experiments hold promise that the development of an effective stress analysis of axially loaded piles is possible.

1.0 INTRODUCTION

During recent years, considerable effort has been expended in improving the analytical methods used to predict the behavior of axially-loaded piles. The efforts have been rather wide-ranging in the degree of complexity, varying from statistical fitting of empirical factors to the results of many pile load tests, Lambda Method (Ref. 1), Improved Alpha Methods (Ref. 2) to the more sophisticated methods based on various theories and concepts of consolidation and shearing resistance of soils (Refs. 3, 4).

In the research devoted to the more sophisticated approaches to pile analysis, the work was performed with only limited experimental data to guide and to calibrate the theories and computer models which were developed. As a result, the procedures which were developed have not been completely successful nor as widely accepted as perhaps could have been possible, had the procedures been guided by a larger base of reliable experimental data, in which the proper soil parameters had been available.

In order to obtain the experimental data by which the various methods could be verified, calibrated, or modified, two in situ tools have been developed at The Earth Technology Corporation. These tools, both instrumented to measure the necessary soil parameters, have now been successfully deployed at the sites of three major pile load tests: the Conoco offshore pile load test site in West Delta Block 58 of the Gulf of Mexico; the Shell Beta pile load test site at Long Beach, California, (Ref. 5); and the Chevron pile load test site at Empire, Louisiana, (Refs. 6 and 7).

Two of the sites contain predominantly soft clays, with the Long Beach site consisting mostly of a stiff silty clay. At each site, direct comparisons can be made between the results of the experiments using the in situ tools and the large-scale pile tests. The results may thus be used to establish relationships among diameter, consolidation, and pile capacity. At the soft clay sites, the behavior of full- and partial-displacement piles can also be compared. It is hoped that the pile segment tools can eventually be used to obtain the necessary design parameters directly.

2.0 OBJECTIVES OF THE RESEARCH PROGRAM

The research program developed and proposed by The Earth Technology Corporation to the industry included an examination of four aspects:

1. The effects of plugging on shear transfer; i.e., a comparison of driven pile models with closed-end and open-end cutting shoes,

2. The effects of diameter on consolidation and shear transfer; i.e., a comparison of the X-probe and the closed-end 3-in. probe,

3. The comparison of the results of the small-scale experiments to the Chevron pile load test; i.e., will a small pile segment model yield the same shear transfer as a larger-scale pile?,

4. The provision of directly obtaining t-z relationships; i.e., by comparing the results of the small-probe experiments to the results of the large-scale pile test, both the shear capacity and the deformational characteristics of the soil may be calibrated against those measured during the Chevron pile study.

The program of research at the Empire site was also expected to provide an expansion of a data base upon which to build a more reliable method for predicting pile capacity, which will include consideration of the effects of diameter, degree of cavity expansion, and soil properties, such as shear strength, permeability, and sensitivity. Comparative data are now available from three sites, two of which have similar soils; future work in other soils will further expand the body of data which may be used to increase the understanding of pile-soil interaction and to guide in the development of reliable design procedures for axially-loaded piles (Ref. 8).

3.0 SITE DESCRIPTION

The test site, shown in Plate 1, is approximately one mile south of Empire, Louisiana. The stratigraphy of the soils at the site in the zone of interest are shown in Plate 2 (after Ref. 6). The shear strength profile at the site, taken from Ref. 6, suggest a normally consolidated clay profile throughout the zone of interest; however, the results of the experimental work show that such a profile is not necessarily accurate. The soil between the depths of 100 and 170 ft is a normally consolidated clay, with reasonably consistent plasticity indices. Although not necessarily obvious from the idealized stratigraphy shown in Plate 2, the soil becomes quite variable with a high degree of interbedding below the 170-ft depth. The interbedded and siltier nature of the soil continues to a depth of approximately 239 ft, upon which the soil again becomes a uniform clay, with reasonably consistent plasticity indices, which, however, are unlike those of the upper clay layer.

It is suggested that the soils were deposited under different geologic conditions, with the coarser particles being deposited near the mouth of the Mississippi River, and the clays being deposited at a time when the main channel of the river was at a different location, so that quiet and/or deeper water conditions existed at the site. Thus, while it is not expected that the clay layers would have an appreciable amount of horizontal variability, the layer between the depths of 170 to 240 ft could be quite variable, both horizontally and vertically.

The location of the test borings which were made during the course of this work are shown in Plate 3. Also shown are the locations of the borings made by other researchers, including the original pile test and soil borings made by McClelland Engineers, the CPT and piezometer work done by Fugro Gulf, and the recent work by the Massachusetts Institute of Technology. In view of the large numbers of earlier borings and probings done at the site, our experiments were conducted 40 ft from the original test pile, in order to avoid placing a boring in, or adjacent to, an earlier one. As previously discussed, horizontal variability in the clay soils was not expected to be significant. This fact was confirmed by the excellent agreement noted during the course of this research. Variability was observed in the soil layer at the 230-ft depth; however, the variability was probably vertical, although there are not sufficient borings upon which to judge the horizontal variability of this layer.

4.0 TEST EQUIPMENT

The testing apparatus used during the course of the experiments consisted of a loading frame upon which a through-hole hydraulic ram was mounted, digital and analog data acquisition systems, and two each of the X-probes and the 3-in. diameter probes.

4.1 Loading Apparatus

The loading frame, shown schematically in Plate 4, consists of two steel plates separated by four steel bars. The lower plate rests on two wooden timbers which serve to elevate the lower plate and to distribute the reaction forces to the ground surface. The upper plate is connected by chains to a number of earth anchors, which provide the necessary reactions for compressive loading. The through-hole hydraulic ram has a capacity of 350 kips at 8,000 psi with a total stroke of 12 in. The hydraulic ram is controlled by one-directional flow-control valves, which are also mounted on the upper steel plate. The direction of travel of the ram is controlled by an electrical solenoid valve, which can be activated either manually, or by micro-switches mounted on an analog recorder, or by a signal from the digital-output from the voltmeter. During the course of the experiments, each type of control was used depending upon the particular load test which was being performed.

In addition to the solenoid valve which controlled the direction of travel, an additional solenoid valve, activated manually, controlled two additional flow-control valves which made up a parallel hydraulic circuit. The second hydraulic circuit was used for fast-rate loading and for rapid adjustment of ram height when first connecting the loading system to the N-rods.

The nominal rate of travel of the ram during the quasi-static load tests and during the plastic-slip portion of the cyclic tests was preset to approximately 0.001 in. per second. The actual rate of travel during the tests varied from 0.0006 to 0.0024 in/sec, with the variation in rate being affected primarily by the degree of precision in repeating the setting of the flow-control valves. The fast loading rate also varied, from a minimum value of 0.011 in/sec to a maximum of 0.059 in/sec. For each test, a direct comparison of the shear transfer at the slow and fast rates was made. In each case the increase in displacement rate covered at least one order of magnitude.

By controlling the rate of travel of the hydraulic ram, the top of the N-rod string was moved at a constant rate of displacement. Due to the accumulation of elastic deformation in the N-rods, the rate of displacement of the probe was constant only during plastic slip under constant shear. For those tests in which the shear either increased or decreased after yield, the rate of displacement of the probe during plastic shear was variable. For those tests in which the shear-displacement behavior showed a definite peak followed by a sharp reduction and then a constant value of residual resistance, the rate of displacement of the probe varied considerably in the immediate post-peak region, until equilibrium had again been achieved between the residual shear force and the amount of elastic deformation in the N-rod string.

4.2 Data Acquisition

The data acquisition system is shown schematically in Plate 5. The system consists of parallel analog and digital data acquisition systems, with each system being used to verify and supplement the other. The digital system was used throughout the course of the experiments to record data during consolidation; both systems were used to record the shear-displacement and pressure-time response during the load tests. Because the sampling rate of the digital system was not fast enough to capture the rapid post-peak t-z data, the analog system was used to provide a complete record.

The analog data acquisition system consisted of a signal-conditioner and amplifier and two x-y-y recorders. The strain-gage signals from the transducers in the 3-in. probes were amplified, filtered, and recorded on the x-y-y recorders. For the X-probe, which has signal-conditioning and amplifying circuitry in the probe itself, only the zero-balance function of the instrument was required. The data recorded during each load test included shear and displacement on one recorder and the total and pore pressure data variations with time on the second recorder.

The digital data acquisition system consists of a Hewlett-Packard 3497A Scanning Digital Voltmeter and a Digital Equipment Corporation PDP 11/23 computer. The HP3497A voltmeter is capable of resolving voltages to a precision of one microvolt; thus, the strain-gage output can be read and recorded directly, without the use of amplifiers or other bridge-balancing circuitry. The use of this system thus eliminates the usual instruments most associated with drift and uncertainty in strain-gage measurements: the signal-conditioners and amplifiers.

The digitized voltages are passed to the DEC computer, which then records the raw voltages on 8-in. floppy discs, converts the voltages to engineering units, and displays the results on the terminal and on a serial line printer. Because of the method used to obtain zero-voltage readings in the field (one sample of each channel, upon demand), the field data logs are only approximate; final data reduction is done after careful study of the data, with averaging of several samples being used, rather than single samples.

Upon the later conversion of data to engineering units, a Hewlett-Packard 9872A Digital Plotter is used to display the test results in graphic form. The digital plotter can be used to quickly plot any set of variables, resulting in reasonably efficient cross-plotting and correlation of the data.

4.3 The 3-In. Diameter Pile Segment Model

A 3-in. diameter probe is shown schematically in Plate 6. The probe is equipped with two load cells separated by a distance of 31.6 in., corresponding to a shaft surface area of two square feet between the load-measuring points. The load cells are connected in a single Wheatstone bridge, so that the output is proportional only to loads applied between the two cross-sections. Located midway between the load cells are a total pressure load cell and a pore pressure transducer. The total-pressure load cell is sensitive only to forces normal to the outer face of the unit, thus measuring the total radial pressure acting against the probe. The face of the transducer is shaped to conform to the pile wall and is of a diameter chosen to have one square inch of surface area. The pore pressure transducer is a commercially-available type manufactured by Gould, Inc. The transducer face forms the rear wall of a cylindrical cavity, with the front face of the cavity being a Carborundum porous stone, thus shielding the transducer from intergranular pressures but allowing free passage of water to transmit pore-water pressure.

A slip-joint arrangement is located at the connection between the probe body and the cutting shoe. The body of an LVDT is mounted on the body of the probe; the core of the LVDT is connected to the cutting shoe. The cutting shoe serves as a reference anchor for the measurement of relative pile-soil displacement, and therefore, enables the direct measurement of shear-displacement (t-z) relationships.

The axial load cells were calibrated by placing a series of loads on each load cell independently using a specially-constructed calibration jig. The calibration factor for the combination of the two load cells was then calculated in order to account for the slight differences in sensitivity. The pore pressure and total pressure transducers were calibrated using hydrostatic water pressure, with a precise Bourdon gage used to determine the applied pressure. In addition, the total pressure load cells were calibrated using a series of dead-weight loads applied through a calibrated proving ring, with the loads then being converted to pressures. The calibration data are plotted, and the conversion factors selected. The data taken during the calibration process were then processed with the same computer program used to reduce the field data, which served to confirm the calibration constants, and to verify the computer program.

4.4 The 1.72-In. Diameter Pile Segment Model (X-Probe)

An X-probe is shown schematically in Plate 7. The probe, which is 56.5 in. in length, contains a total pressure transducer, a pore pressure transducer, a shear-sensing element and a displacement transducer. In a manner similar to the 3-in. diameter probe, the total pressure transducer measures the total force acting on an active face which is shaped to conform to the outside surface of the probe. The pore pressure transducer is mounted in a housing which has an inner cavity connected to the outer surface of the probe through three holes. At the probe surface, a milled hole 3/16-in. deep and 5/8 in. in diameter intersects each of the three paths to the probe interior. Porous Carborundum stones are then pressed into the holes, and their outer surfaces are then shaped to conform to the outer surface of the probe. The shear-sensing element consists of a cylindrical sleeve which is supported by a load cell. The outer sleeve, which is not connected to the probe except through the load cell, thus gives a direct measure of the average shear transfer on the 31-sq. in. surface area of the sleeve.

The X-probe is calibrated in a specially designed cylindrical pressure chamber. An annular sleeve, with O-ring seals, is mounted on the probe. When the probe and sleeve are inserted into the pressure chamber, the annular sleeve forms a hydraulic piston. The body of the X-probe is restrained from motion at each end, and pressure is sequentially applied to each cavity thus formed in the cylinder. The applied pressure is then converted to equivalent shear by using the ratio of the two areas, the sleeve area and the piston area, to determine the calibration factor. At the same time, the total pressure and pore pressure transducers are read and calibrated. The applied pressures are read on a precise Bourdon gage. As with the 3-in. probe, all calibration data are hand-plotted, the calibration constants determined, and the calibration data then cycled through the data reduction computer program to verify the program and the calibration constants.

4.5 Field Operations

Operations on-going at the test site during a portion of the work are shown in Plate 8. The load frame is shown in position for a test in the background, with the drill rig readying another hole for installation of a probe. The third hole may be seen in the foreground. Because of the length of time required for consolidation of the soil after installation of the 3-in. probes, experiments were performed simultaneously in three borings. The data acquisition computer program had been written for such simultaneous experiments; each set of four data channels devoted to a separate probe could be independently sampled, each at a different rate. Only one probe can be actively loaded at any time; however, consolidation data from the other probes can be recorded simultaneously.

The load frame and hydraulic ram are shown in position for a load test in Plate 9. The load head, which is a circular plate welded to a short length of N-rod, can be seen passing through the ram, and attached to the string of N-rods which are used to load the probe.

5.0 TEST PROCEDURES

Prior to mobilizing to the test site, a sequence of experiments was developed which would maximize the productivity of both the drilling crew and the testing crew, and which would yield the maximum amount of data from each experiment, in terms of technical content. Based on earlier experience in similar soils, a degree of consolidation of approximately 50 percent was expected to require consolidation times of 4 and 12 hours for the X-probe and the 3-in. probes, respectively, while a 90 percent degree of consolidation was expected to require 24 and 72 hrs. As will be shown later, the estimates of consolidation time were reasonably close to those observed. The actual degree of consolidation is not of consequence per se; however, in order to obtain repeatability among the experiments, it was required that the consolidation times be the same for each probe type at each depth, and that the 3-in. and X-probes be tested at the same stage in the consolidation process.

For each experiment at each depth, a sequence of installation and load testing was also predetermined, based primarily on the desire to obtain the most information possible from each experiment, in terms of technical content. The procedures which were followed are listed below:

1. Install the probes either by driving (3-in. probes) or by pushing (X-probe).

2. Perform a test to failure, in both tension and compression, as soon as possible after installation. In the clay soils, the tests were performed at degrees of consolidation ranging from 5 to 20 percent corresponding to times ranging from 5 to 15 minutes after installation.

3. Observe consolidation, allowing the process to proceed for the predetermined length of time for each experiment.

4. Perform a slow loading to failure in tension.

5. Allow the pore pressures generated during the initial test to return to those recorded prior to the test.

6. Based on the results of the initial test, perform a series of one-way cyclic tension tests, each with a bias tension load of 50 percent of the failure load, and with a cyclic component above and below the bias load being progressively increased in increments of 10 percent until failure was indicated by progressive upward displacement on each cycle of loading.

7. Perform a slow loading to failure in tension, to determine the effects of the cyclic tension tests on the static behavior.

8. Perform a slow-rate, two-way cyclic load test, with failure in both tension and compression, to obtain the maximum degree of cyclic degradation.

9. With the same displacement limits (increased for some tests), increase the loading rate to the maximum possible, within the limitations of the hydraulic system.

10. Reduce the rate of loading, and repeat the slow-rate, two-way cyclic tests to observe any additional cyclic degradation which occurred during the fast-rate test, and to obtain values of shear transfer to compare with the results of the fast-rate tests.

11. Allow consolidation to proceed following the tests, for as long as possible, in order to observe the nature of the recovery in resistance following the cyclic degradation.

12. Perform a slow loading to failure in tension, with one cycle of loading to failure in compression, immediately prior to removing the probe.

The sequence of tests during each experiment were intended to model, as closely as possible, the behavior of a segment of a long, elastic pile subjected to cyclic loading, with behavior ranging from linear, under very small components of load as experienced near the pile tip to highly nonlinear as experienced in upper portions of a pile where one-directional load cycling of the pile top will produce two-way cycling in the soil due to elastic pile recovery. With some exceptions, which are noted in the discussion of each test, the testing program outlined above was followed during each experiment.

6.0 PRESENTATION OF TEST RESULTS

A brief overview of the research program and selected tests results will be given in this report. Complete records of all the data taken during the work is contained in Volume II. The data shown in this report is intended to serve as a summary of the most important findings during the project, and, as such, is a fairly complete summary of the data. For those who desire more information than is contained herein, it is suggested that a parallel review of both documents be undertaken.

A summary of the experiments performed at the Empire site is shown schematically in Plate 10. As shown in the figure, a total of 14 experiments were performed, at 6 depths. In addition to the work proposed, one additional experiment was performed with a 3-in. probe, at the 210-ft depth, and two additional experiments were performed with the X-probe, at the 141-ft and the 230-ft depths. The additional X-probe experiment at the 230-ft depth was necessary for technical reasons; the remaining extra experiments were performed for completeness.

A summary of the X-probe experiments is given in Plate 11, which shows the value of the measured pore pressures during each sequence of load tests. The data shown in Plate 11 is also given in Table 1. Each symbol in the figure represents the pore pressures measured prior to a load test, although not necessarily related to an independent installation of the probe.

The maximum values of pore pressure are those measured immediately after installation. The ambient pore pressure lines were inferred from the actual measurements made at the 210 and 230-ft depths and an assumed gradient above the 210-ft depth. A total of 21 load tests were performed with the X-probes, each load test being performed at a different degree of consolidation for each probe at each depth. The intermediate lines are drawn at equidistant values of pressure, in order to give an indication of the degree of consolidation at which the tests were performed.

The degree of consolidation for each test was defined as the percentage of the initial excess pore pressure which had dissipated at the time of the test, as

$$\frac{U_{max} - U_{test}}{U_{max} - U_{ambient}}$$

Although this definition does not conform exactly to that used in conventional soil mechanics, it is a convenient parameter by which the results of the experiments can be compared at various stages in the consolidation process.

A summary of the experiments performed with the 3-in. diameter probes is shown in Plate 12, and also included in Table 1. Except for the symbols shown at the maximum value of excess pore pressure at each depth, each symbol represents the pore pressure at the beginning of a load test. Thus, as shown in the figure, eight load tests were performed with the closed-end 3-in. probe, and six load tests were performed with the open-end 3-in. probe.

The definition of the terminology used in describing the values of shear transfer in this report are given in Plate 13. The plate contains the results of the initial two-way cyclic tests performed on the X-probe during the experiment at the 120-ft depth, but is typical of all such tests with the X-probe in clay soils.

The maximum value of shear transfer obtained on the initial loading to failure is defined as the <u>initial maximum</u> shear transfer. After reversal of loading, the shear-displacement behavior observed during the cyclic X-probe tests changed, showing a definite <u>peak</u> followed by a reduced value in the shear with continued displacement denoted as <u>residual shear</u> transfer. Also shown in the plate is the value of the <u>cyclic minimum</u> shear transfer.

6.1 Test Results at the 120-Ft Depth

A summary of the data taken during periods of consolidation during the X-probe tests at the 120-ft depth is given in Plate 14. The variation in the measured radial total pressure and pore pressure are shown in the upper graph. The values of radial effective pressure, which were obtained by subtracting the measured pore pressures from the measured total pressures, are shown in the lower graph.

Values for the pressures are not shown for periods of active loading. These included the immediate test, performed approximately 12 minutes after installation, and the major series of static and cyclic tests, performed about 24 hours (1,440 minutes) after installation.

As seen in the figures, the total radial pressure decreased with time, as did the excess pore pressures, with the form of the pore pressure dissipation curve being very similar to those obtained from ordinary one-dimensional consolidation tests. Unlike the behavior in one-dimensional consolidation, however, the radial effective pressure increases linearly with the logarithm of time, rather than being the mirror image of the pore pressure curve, which would occur had the total pressure not decreased with time.

As noted in the figures, the series of load tests reduced the radial total pressure and increased the pore pressure. The combined changes in the two pressures resulted in a decrease in the radial effective pressure. Upon the completion of the planned series of load tests, data was recorded during a period of several hours, during which the total pressure remained constant, the excess pore pressures generated during the tests dissipated, and the radial effective pressure approached the value which was recorded prior to the series of load tests. At this time, the probe was again loaded to failure in both tension and compression (with no cyclic loading) and then removed.

For this Volume I, the variation in the total pressure with time is included for the 120-ft depth only. For subsequent depths, the pressure data from all experiments at each depth were placed on the same figure. Since the curves showing the changes in total pressure with time would have crossed the pore pressure dissipation curves, the total pressure curves were omitted in order to avoid confusion. Complete records of the time-variation of all three pressures are included in Volume II, where the results of each experiment with each probe are presented separately.

The results of three of the load tests to failure are given in Plate 15. The tests include the initial test to failure after consolidation, a test to failure upon completion of the one-way cyclic tests in tension, and the last loading to failure in tension at the end of the two-way cyclic tests. The data shown in the figure were recorded using the digital data acquisition system. The digital data was used for convenience, so that the curves could be presented in the same figure, to the same scale.

During the initial loading of the X-probe, the slip-joint connection between the probe and the LVDT anchor slipped, causing a small shift in the recorded displacement. This is seen as a discontinuity in the initial loading curve. The data was not corrected, and is shown as-recorded.

The second shear-displacement curve, recorded after the one-way cyclic tension tests, follows a loading path which is almost parallel with the first curve, deviating only near yield. During the post-yield plastic slip, the shear transfer equalled the value which was recorded prior to the series of one-way cyclic tension tests, indicating that the application of many cycles of one-way loading in tension did not result in any cyclic degradation in the resistance. Similar results were observed in all the cyclic tension tests in clay; that is, without reversal of plastic deformation and slip, no loss was observed.

The next, lower curve is one which was recorded during the two-way cyclic tests. In almost all the tests in clay, the reversal of loading produced a change in the nature of the t-z response. From a response which was at first essentially elastic-plastic, the curve shape changed to that of a strain softening material, in which the resistance decreased sharply after a limiting resistance has been attained. Until the direction of loading is again reversed, the lower residual resistance becomes the maximum value that can be reached, even at the end of a cycle of unloading and reloading, but without any reversal of slip.

Because the loading rate was controlled by applying a fixed rate of displacement at the top of the N-rod string, the displacement of the probe downhole, immediately after the yield displacement was reached, was not controlled. The string of N-rods had accumulated an amount of elastic strain energy, which was stored as elastic deformation. As the yield point was reached, and the shear decreased with increasing shear, until failure. During plastic slip, little change occurred in either the shear or the effective pressure. The curve shown includes only the values up to peak, in order to produce a cleaner figure.

Upon the subsequent loadings to failure, the stress path showed a tendency to bend to the right during plastic slip, as the radial effective pressure increased, with either increases or decreases in the shear transfer. Such behavior is observed in ordinary laboratory shear tests on samples of overconsolidated clays which dilate during shear.

It should be noted, however, that the stress paths obtained from the pile segment models are not the exact equivalent of p-q diagrams obtained from laboratory triaxial tests. In the triaxial apparatus, the shear stress is computed from differences in the normal stresses. Since the stresses in the three orthogonal directions are known, the shear stress will always have a known relationship to the principal stresses.

In the stress paths shown here, the normal stress (or pressure) is known for only one direction. In addition, the measured shear transfer is not exactly the shear stress on the failure surface, but rather an equivalent shear stress which would exist at the pile-soil interface, if slip occurred there. In all the experiments in clay, a thin layer of soil adhered to the probe, and remained on the probe after removal from the boring. The adherence of a clay layer on the probe clearly shows that the shear failure is on a soil-soil surface, and that the measured shear transfer is not exactly the failure stress existing on the failure surface.

The measured total radial pressures are the confining pressures from the soil outside the slip surface; the variations in the total radial pressures are responses to volumetric changes in the cylindrical shear zone. Since the failure occurs at some distance from the probe surface, the pore pressures are also not exactly those which may exit at the failure surface.

Due to the combination of factors described above, it can be seen that an exact correspondence between the stress paths shown and those from laboratory tests should not be expected. During consolidation, when the pressures are in a reasonable condition of equilibrium, the values of effective pressure calculated from differences in the total and pore pressures are probably quite good. During loading, however, the stress paths should be used in a qualitative sense, without too much emphasis on the calculated values, which are not those in the soil inside the shear zone as previously discussed.

It can be noted in the lower two figures of Plate 16 that during cyclic tests, failure occurs at a value of radial effective pressure that is less than the initial consolidation pressure, shown in the top figure, suggesting that the behavior should be that of an overconsolidated clay. If this interpretation of the stress path is correct, then the value of maximum shear transfer may not have a simple relationship to the radial effective pressure, this is also known to be the case with overconsolidated clays during undrained loading. This is not to imply, however, that the value of maximum shear transfer is not related to the value of the mean normal effective stress or radial effective pressure during consolidation.

6.2 Test Results at the 141-Ft Depth

A summary of the consolidation data recorded at the 141-ft depth is shown in Plate 17. The figure contains data from two X-probe tests and from two 3-in. probe tests. As expected, higher values of excess pore pressure were developed by the full-displacement probes, but with reasonable agreement among the three. During consolidation, the behavior for the two X-probes is almost identical, for both the pore pressure and the radial effective pressure. The horizontal separation between the consolidation curves for the X-probes and the closed-end 3-in. probe should also be expected; the increase in consolidation time is due to the difference in diameter. The parallel nature of the curves indicates very similar behavior among the three full-displacement probes, indicative of uniformity in the clay layer at this depth and validity of the experiments.

The lower pressures recorded for the open-end 3-in. probe were also expected. Since the probe is, in effect, a "cookie-cutter", there should be a lower degree of cavity expansion and, therefore, a smaller amount of excess pressure generated. The pressure-time curve also differs for the partial-displacement probe in that the consolidation occurs more rapidly than for either of the full-displacement probes.

The rate of development of radial effective pressure also differs for the two types of probes. The partial-displacement probe shows a diminishing rate of increase in effective pressure with the logarithm of time, while the full-displacement probes show parallel continuous and almost linear, increases.

The difference in the rates of consolidation between the two 3-in. probes had not been foreseen. As a result, the initial load tests performed for the X-probes and the full-displacement 3-in. probe were performed at degrees of consolidation of 52, 47, and 48 percent, respectively, near the intended value of 50 percent. However, the partial-displacement pile, with the same consolidation time, had attained a degree of consolidation of 89 percent, far in excess of the intended 50 percent. This difference in the degree of consolidation at the time of the tests produced a predictable result; the partial-displacement pile recorded a higher maximum shear transfer of 1.04 ksf, as compared to values of 0.76, 0.84, and 0.76 ksf for the two X-probes and the 3-in. full-displacement pile, respectively. After an additional period of consolidation, during which the three full-displacement probes reached values of 76, 75, and 75 percent of consolidation, the resistances increased to 1.28, 1.11, and 1.21 ksf, respectively. These values are in closer agreement with those recorded for the partial-displacement probe at a similar degree of consolidation.

The results of the load tests on one of the X-probes are given in Plate 18. The increases in resistance shown for the tests after the cyclic tension tests are a natural consequence of the consolidation which occurred during tests. The initial load test was performed after four hours of consolidation; the cyclic tension tests began 5 hrs after installation, and took 3 additional hours to complete. The cyclic tension tests did not create sufficient disturbance and plastic shear to interrupt the consolidation process; thus, the maximum shear transfer continued to increase throughout the cyclic tension tests. After this first experiment using the X-probe, the cycle times and numbers of cycles were decreased.

As was observed in the experiment at the 120-ft depth, the effect of the two-way cyclic tests was a reduction in the plastic shear resistance. Again, the t-z behavior changed from an essentially elastic-plastic response to a brittle, peak-residual pattern. The discontinuous t-z curve recorded by the digital system is again shown, in order to present the test results to the same scale on the same figure. The complete analog curve is shown in Volume II. The downward slope of the curve after yield is, again, a consequence of variable load-rate effects. Had the displacement of the probe been controlled down-hole, the soil would have exhibited a more-normal peak-residual form, with a constant value of residual resistance being reached quickly after the peak.

The maximum shear resistance, recorded after the cyclic tension tests, was 0.97 ksf. During the two-way cyclic tests, the peak resistance was 0.93 ksf, with a residual value of 0.70 ksf.

The effects of cyclic loading are similar to those observed at the 120-ft depth. The two-way cyclic loading again resulted in a reduced stiffness of the curve; the result of a simultaneous reduction in the limiting shear and an increase in the yield displacement. This indicates that the clay surrounding the shear zone has also been affected by the cyclic shearing since, had the clay not been softened, the yield displacement would have simultaneously decreased with the decreases in the yield stress.

The stress paths followed by the probe during the three load tests are shown in Plate 19. The behavior is much like that exhibited at the 120-ft depth, with a trend for the radial effective pressure to decrease during the initial loading to failure, with little change during plastic deformation. Again, the radial effective pressure increases during plastic deformation upon subsequent loading to failure.

The results of the load tests on the 3-in. diameter full-displacement probe are given in Plates 20 and 21. During the initial test series with this probe, the resistance of the probe in compression exceeded the capacity of the earth anchors, pulling them out of the ground. The two-way cyclic tests were postponed for one full day, while additional earth anchors were obtained and installed.

As seen in Plate 20, the cyclic tension tests had little effect on the maximum shear transfer, but did change the shape of the curve slightly, softening the early portion and rounding the curve near yield. Although the degree of change in the behavior would have little practical significance in actual pile design or analysis, the behavior again suggests a weakening of the clay outside the shear zone with degradation not being confined only to the slip surface.

One unanticipated benefit of the failure of the earth anchors was the additional consolidation which occurred during the 24-hr waiting period. During this time, the consolidation progressed to 75 percent, much nearer the 89 percent of the partial-displacement probe. As shown in Plate 21, the maximum shear transfer increased to a value of 1.21 ksf. Since little could be gained from another series of cyclic tension tests, the probe was immediately loaded to failure in compression, and the sequence of two-way cyclic loads were applied. The two-way cycling resulted in a decrease in shear transfer to 0.77 ksf, a loss of more than 36 percent. The t-z curve was also altered in shape, becoming more like the peak-residual shape recorded for the X-probe, but with a less pronounced decrease after yield.

It is also of interest to note again that the yield displacement has not been changed by the cyclic loading. The surrounding soil (in which the pre-yield deformation occurs) has thus again been softened, in addition to the cyclic degradation occurring along the slip surface.

The stress paths followed by the soil during three of the load tests given in Plates 20 and 21 are shown in Plate 22. The stress paths are for the two initial load tests and for a portion of the final cycle of the two-way cyclic load test. As seen in the figure, the behavior is reasonably consistent during the three loadings, with differences appearing only in the post-yield region.

The results of the load tests on the open-end 3-in. probe are shown in Plate 23. This was the first experiment performed at the site and was, therefore, used to determine the limits of behavior; that is, the values of maximum and cyclic minimum shear transfer which would be used to plan the cyclic tension tests during subsequent experiments.

As shown in the figure, the maximum shear transfer was 1.04 ksf on the initial loading. The pore pressures were then allowed to recover, at which time the two-way cyclic test was performed. The maximum resistance on the next loading to failure was 1.08 ksf, which then reduced to 0.78 ksf after several cycles. The effects on the shear-displacement behavior are similar to those exhibited by the full-displacement probes that is a softening of the early portion of the curve, with a characteristic peak-residual shape after yield. Again, the losses in shear resistance occurred with little change in the yield displacement.

The stress paths followed by the soil during the three load tests are shown in Plate 24. Again, the behavior is consistent among the three tests and resembles that exhibited by the X-probe more closely than that observed for the full-displacement 3-in. probe.

Although there is some disagreement in the values of maximum shear among the three probes, the agreement is much better among the three which were tested at the same degree of consolidation, being 0.76 ksf, 0.84 ksf, and 0.76 ksf. Better agreement is also shown between the two 3-in. piles during the two-way cyclic test, with cyclic minimum resistances of 0.78 ksf for the partial-displacement probe and 0.77 ksf for the full-displacement probe.

6.3 Test Results at the 160-Ft Depth

A summary of the consolidation data recorded at the 160-ft depth is shown in Plate 25. Much like the behavior observed at the 141-ft depth, the full-displacement probes create almost the same initial excess pore pressure. The dissipation of the excess pore pressures with time is also similar, with the curves being separated by a time constant which is related to the differences in diameter. The open-end probe, on the other hand, again creates smaller excess pore pressures, with, again, a more rapid time-rate of dissipation.

One aspect of the differences in the behavior of the full-displacement probes as compared with the partial-displacement probe is the development of radial effective pressure during consolidation. The partial-displacement probe shows a diminishing rate of increase in radial effective pressure; the full-displacement probes, with higher pressures, are still exhibiting linear increases with the logarithm of time.

The results of the load tests on the X-probe are shown in Plate 26. As noted earlier, the cyclic tension tests had little effect on the shear transfer, either with regard to magnitude or to stiffness. The two-way cyclic tests, on the other hand, again resulted in the behavior becoming distinctly peak-residual in character, with the residual resistance being reduced to 0.93 ksf, as compared with 1.18 ksf on the initial loading and on the loading after the cyclic tension tests.

The stress paths followed by the soil during the three load tests are shown in Plate 27. The pattern of behavior is, again, very similar to that exhibited in the experiments at the 120- and 141-ft depths, indicating that the clay was, indeed, very uniform in properties with depth.

The results of the load tests on the full-displacement 3-in. probe are shown in Plate 28. During the initial loading, the t-z behavior follows a peak-residual behavior pattern. Only the postion of the curve up to the peak value of shear was captured adequately by the digital system. The analog curve is shown in its entirety in Volume II. The effects of the cyclic tension tests were a stiffening of the shear-displacement curve; the magnitude of the limiting resistance was not changed, however. After the two-way cyclic tests, the shear-displacement curve has a significantly lower limiting resistance; however, the yield displacement has again remained nearly the same.

The peak resistance during the initial loading (taken from the analog record) was 1.48 ksf; that following the cyclic tension tests was 1.49 ksf. During the two-way cyclic tests, the limiting shear resistance was reduced to 0.89 ksf, a reduction of 40 percent.

The stress paths followed by the full-displacement 3-in. probe during the three load tests are shown in Plate 29. The behavior is similar to that exhibited at the 141-ft depth, with a tendency for the radial effective pressure to decrease during loading, with decreases also observed during the plastic deformation. Only minor changes in the effective pressure are shown during plastic deformation during the remaining tests.

The results of the load tests on the partial-displacement probe at the 160-ft depth are shown in Plate 30. The slight discontinuity in the initial shear-displacement curve is due to a temporary slippage of the LVDT. Again, the one-way cyclic tension tests had little effect on the static response; the limiting shear transfer after the cyclic tests was 1.38 ksf, as compared with 1.35 ksf recorded on the initial loading. Two-way cyclic loading again showed significant effects on the shear-displacement behavior, softening the pre-yield portion of the curve and reducing the limiting shear transfer to 0.92 ksf. The yield displacement was not greatly affected, however.

The stress paths followed during the three load tests are given in Plate 31. As with the other probes, the behavior is consistent with that observed in the experiments at the 141-ft depth.

It is of interest to note that, at this depth, the degree of consolidation at the time of testing was much closer for the three probes, being 84 percent for the X-probe, 83 percent for the full-displacement 3-in. probe, and 92 percent for the partial-displacement 3-in. probe. The initial limiting shear transfer values were also similar, being 1.18 ksf, 1.48 ksf, and 1.35 ksf, respectively. The cyclic minimum shear transfer values were even in closer agreement, being 0.93 ksf, 0.89 ksf and 0.92 ksf, respectively.

6.4 Test Results at the 210-Ft Depth

A summary of the consolidation data from the three experiments at the 210-ft depth are given in Plate 32. The behavior observed at this depth is quite different to that recorded in the clay layer between the depths of 120 and 160 ft. Consolidation for all three probes was very rapid, being essentially complete in approximately 2 hrs for the 3-in. probes, and in 4 hours for the X-probe. The consolidation behavior indicates that the material is granular in nature, with a much higher permeability than the clay in the upper layer. As noted in the boring log, the soils between the depths of 170 and 240 ft were very nonuniform; the results of the experiments at this depth verify that the soils were, indeed, primarily granular soils with low cohesion.

The development of radial effective pressure with time also differed considerably among the three probes. The X-probe has a 60-degree conical tip, and was pushed into place; the full-displacement 3-in. probe has a flat end, and was driven into place. The combination of differences in shape and in the method of installation is probably responsible for the large differences in behavior shown between the two.

It is also of interest to note the very low lateral effective pressure created by the installation of the partial-displacement probe. The values shown are very much lower than those for the other probes, and are even smaller than those recorded at the 141-ft depth using the same probe.

The results of the load test program on the X-probe are shown in Plate 33. Unlike the earlier tests, the cyclic tension tests reduced the limiting shear somewhat, from 1.94 ksf to 1.80 ksf. It is also of interest to note that the resistance measured during an immediate test within minutes after pushing the probe into place was 2.03 ksf, greater than that observed at the end of consolidation. Two-way cycling also had less effect, with the change in shape from elastic-plastic to the peak-residual form, but with no reduction in the peak resistance. The residual resistance, however, was reduced to 1.49 ksf.

The stress paths followed by the soil during the three load tests are shown in Plate 34. As would be expected, the behavior observed during loading differs from that shown for the clay layer between the depths of 120 and 160 ft. Perhaps of most interest is the post-yield behavior during which significant increases in the radial effective pressure accompany the plastic deformation under constant shear transfer.

The results of the load tests on the full-displacement 3-in. probe at the 210-ft depth are shown in Plate 35. The behavior differs markedly from that previously observed, with a distinct stick-slip character. The soil would yield, then the probe would essentially stop, the load would increase, and another plateau at yield would follow. This behavior continued throughout the experiment, as shown in the lower curve, which was recorded after the two-way cyclic tests. The stick-slip response is, again, a consequence of the method by which the loads were applied. During the increases in shear after each yield plateau, the elastic deformation of the N-rod string had to increase in proportion to the load. The upward movement of the probe was thus temporarily arrested until the hydraulic ram had accommodated the additional elastic deformation in the system.

The maximum resistance recorded for the probe on the initial loading was 1.78 ksf. After two-way cyclic tests, the first yield plateau was reached at a value of 0.83 ksf, the second plateau at 0.91 ksf and the resistance continued to increase to a value of 1.05 ksf.

The stress paths followed during the initial failure loading and on the final cycle of the two-way cyclic tests are shown in Plate 36. Also shown is the behavior observed during the removal of the probe, during which the rate of displacement was varied after failure. The behavior is again unlike that observed in the clay soil, and also unlike that observed for the X-probe. A comparison of the two upper graphs shows that, although this soil was probably cohesionless, the reduction in resistance during two-way cycling was not directly related to losses in radial effective pressure.

The behavior exhibited during the variable-rate test, shown in the third graph, is quite interesting. During the initial rate increase, the shear transfer increases with little change in the radial effective pressure. When the slip rate is reduced, with the accompanying reduction in the shear transfer, the radial effective pressure shows a significant increase.

The results of the load tests on the partial-displacement 3-in. probe are given in Plate 37. The results of the test are, again, dissimilar to those observed in the clay layer. The peak resistance on the initial loading is 0.95 ksf. After the one-way cyclic tension tests, the limiting resistance was very nearly the same at 0.97 ksf. After two-way cycling, the limiting shear had been reduced to 0.69 ksf, a reduction of almost 30 percent.

The stress paths followed during the three load tests are shown in Plate 38. The behavior again differs from that observed for the full-displacement probes, with no tendency for the radial effective pressure to increase during plastic deformation.

The substantial differences in the response of the three probes at this depth, both during loading and during consolidation, indicate that the soil was not a clay. A high degree of layering was also noted during the insertion of the X-probe, with a wide variation in the force which was required to advance the probe.

As previously mentioned, a continuous film of clay, varying in thickness from 1/8 to 1/4-in., was always found adhering to the probes after removal from the upper clay layers. Examples of the clay films which adhered to the 3-in. probes in the upper clay layer are shown in Plates 39 and 40. At the 210-ft depth, the layers of consolidated clay shown in Plates 39 and 40 were absent, and upon removal of the probes, only spots and patches of soil adhered to the probes.

6.5 Test Results at the 230-Ft Depth

Consolidation data taken in two borings by two X-probes at depths of 229.5 and 230 ft are given in Plate 41. During the installation of the X-probe at 229.5 ft, two problems arose. The excess pore pressures which were developed resulted in an output which exceeded the capacity of the strain-gage amplifier. The X-probe tip also encountered a dense layer of sand, which could not be penetrated by pushing. In order to obtain measurements of the maximum pore pressures after installation, the experiment was repeated in another boring, with the amplifier gain reduced and with the 6-in. extension removed from the probe tip. As seen in the figure, moving the probe about 20 ft horizontally and 6 in. vertically resulted in a considerable difference in the soil type being reached. Although similar maximum total and excess pore pressures were developed, there was a significant difference in the rate of dissipation of excess pore pressure with time. Since the probes are identical in diameter, the time difference in consolidation can only be due to differences in the permeability and compressibility of the two materials.

The difference in the two materials was evident when the probes were removed from the boring. The probe at the 229.5-ft depth was in a silty clay, with a 1/8-in. thick layer of soil adhering to the surface of the probe after retrieval, as shown in Plate 42. The probe which was pushed to the 230-ft depth was retrieved in an almost-clean condition, with only spots of clay adhering to the probe, as shown in Plate 43.

A 4-hr period of time was allowed for consolidation, which produced a degree of consolidation of 56 percent, close to the value of 50 percent which was expected.

The probe was then loaded to failure in tension, and produced the results shown in Plate 44. Because of the problems encountered during insertion of the probe at the 229-ft depth, the one-way cyclic testing program was omitted and the probe was directly subjected to a number of cycles of two-way loading, which, again, resulted in the shear-displacement response becoming of a peak-residual form. The complete test results are given in Volume II; only the portion recorded by the digital system is shown here, in order to compare the behavior. The maximum resistance on the first loading was 2.06 ksf; the residual resistance after two-way cycling was reduced to 1.75 ksf.

The stress paths followed during the two load tests are given in Plate 45. The behavior shown in the two upper graphs is similar to that observed in the upper clay layer. In addition, the stress path followed during a load test to failure after an additional 12 hrs of consolidation is shown in the lowest graph. During this test, the rate of displacement of the probe was increased from 0.001 in./sec at yield to 0.026 in./sec during plastic slip. The soil exhibited a 30 percent increase in resistance, with the radial effective pressure almost doubling.

The shear-displacement behavior recorded during the load tests at the 230-ft depth are shown in Plate 46. Much like the behavior observed for the full-displacement 3-in. probe at the 210-ft depth, the shear-displacement response displayed a stick-slip pattern, with yielding immediately followed by a reduction in load. The maximum resistance reached at each peak remained fairly constant, varying from 2.35 to 2.38 ksf.

Because of the unexpected nature of the response, the test program was revised. Attempts were made to cycle the probe in tension and compression, but were not successful. During the compression loading, the N-rods buckled against the casing, preventing the probe from attaining soil failure in compression.

Following the abortive attempts at failing the probe in compression, the soil was allowed to continue to consolidate for a period of 12 hrs. The probe was then loaded to failure in tension, with the maximum resistance being increased to 2.88 ksf.

The stress paths followed during the load tests are shown in Plate 47. The behavior most resembles that recorded for the full-displacement 3-in. probe at the 210-ft depth.

6.6 Results of the Tests at the 250-Ft Depth

A few days earlier, the power pole at the site had fallen, and was resting against the roof of the data acquisition building. The Louisiana Power and Light Co., after discovering the situation, had allowed the electrical power to be used only until 4 p.m that day. In order to complete the tests by shut-down time, it was decided to not wait for further consolidation. The load tests were therefore begun at 1:30 pm.

The data recorded during consolidation at the 250-ft depth is shown in Plate 48. The consolidation curve indicates that the probe had, at this depth, encountered a clay. The soil had been allowed to consolidate for 26 hrs, at which time the degree of consolidation had reached 77 percent.

The results of the load tests are shown in Plate 49. The maximum resistance on the initial loading was 1.76 ksf. The one-way cyclic tension tests were begun; however, after a few cycles, problems were encountered with one of the flow-control valves. Since there was not sufficient time to replace the valve and complete the test program, the one-way cyclic tension tests had to be discontinued, and the two way cyclic tests were begun.

The results of the first loading to failure after the one-way cyclic tension tests are included in Plate 49. The static behavior was not changed appreciably, with a maximum resistance of 1.73 ksf being recorded. The cyclic minimum shear transfer during the two-way cyclic test was 1.12 ksf, with the familiar peak-residual form. This reduction in resistance, of 36 percent, is similar to that recorded in the upper clay layer.

The effects of the cyclic loading on the form of the shear-displacement behavior is similar to that recorded in the upper clay layer; reductions in the limiting shear were experienced without much effect on the yield displacement.

The stress paths followed during the three load tests are given in Plate 50. The behavior exhibited at this depth also followed the form of the response observed in the upper clay layer.

7.0 DISCUSSION OF TEST RESULTS

The purposes of this report are to present the results of the experiments at the Empire site, to demonstrate the capabilities of the tools which have been developed, and to demonstrate the utility of the tools in obtaining the soil parameters necessary for axial pile design and analysis.

At this time, it is not felt that the presently-available theories of axial pile behavior nor the published analytical methods fully explain all aspects of the behavior which was recorded during the experiments. The experiments will, however, add a considerable amount of data which will prove valuable in the future development of a rational theory and analytical model of axial pile behavior.

The discussion of the test results will be limited to a presentation of the trends observed in the experiments and a comparison of the results with those published for the earlier pile load tests at the site. Most attention will be directed to the experiments performed in the clay layer. Insufficient information is available regarding the soil types and properties at the lower depths from which to draw any firm conclusions relating the observed behavior to the soil characteristics.

7.1 Summary of Measured Lateral Pressures

A summary of the maximum total pressures recorded immediately after installation of the probes is shown in Plate 51. It can be seen in the figure that excellent agreement was obtained among the maximum pressures measured by all the full-displacement probes at all depths. The maximum total radial pressure increased linearly with depth, except for those measured in the layer of soil at the 210-ft depth.

The maximum total radial pressures measured after installation of the partial-displacement probe are smaller in magnitude than those created by insertion of the full-displacement probes, but maintain the same relative proportion with depth.

The maximum pore pressures which were measured immediately after installation of the probes are shown in Plate 52. As with the maximum total pressures, the excess pore pressures created by the full-displacement probes in each soil layer increase linearly with depth. The maximum excess pore pressures measured for the partial-displacement probe also vary linearly with depth but are, again, less in magnitude than those recorded for the full-displacement probes.

The magnitudes of the excess pore pressures generated by the full-displacement probes in the clay layer between the 120- and the 160-ft depths were approximately 10 times the undrained shear strengths given in the interpreted shear strength profile in Ref. 6. For the partial displacement probe, the excess pore pressures were approximately equal to 8 times the undrained shear strength values.

At depths below 160 ft, the maximum excess pore pressures created by the full-displacement probes again vary linearly with depth, but not on a continuation of the same trend line as in the upper layer. Had the site been a uniform normally consolidated clay deposit, proportionality should have been maintained throughout the depth of the profile. The decrease in the magnitude of the excess pore pressures with respect to the total pressures in the lower layer is due to differing permeability and compressibility characteristics of the soils. As indicated by the rather significant differences in the consolidation characteristics of the soils at the 210- and 230-ft depths, the soil at these depths was not a uniform clay, so that direct comparisons of the magnitude of the excess pore pressures with those measured in the upper clay layer are probably of little value.

The two lines shown on the figure as ambient pore pressure lines were obtained from measured values of pore pressure at the 210- and 230-ft levels. At the 210-ft depth where the consolidation was completed in hours, the pore pressures recorded for the partial-displacement probe remained constant (±0.02 ksf) for a period of 3.5 days (except for an active loading period). It was therefore assumed that all ambient pressures above this depth included a normal excess pressure of 0.87 ksf (approximately 13.6 ft of sea water). At the 230-ft depth, the probe which was in the interbedded material reached a value of pore pressure of 14.61 ksf, which also showed no further decrease with time. This value is consistent with a water depth of 230 ft, which indicates the absence of excess pore pressures in the lower clay layer.

7.2 Limiting Values of Shear Transfer

The variation with depth of the peak shear transfer, or skin friction, is given in Plate 53. As seen in the figure, reasonable agreement is obtained among the values of peak resistance among all the probes in the clay layer between the depths of 120 and 160 ft at comparable degrees of consolidation. The values also agree reasonably well with the average value reported in Refs. 6 and 7 for the 14-in. diameter pile tested between the depths of 115 and 165 ft.

The variability in the soil at the 210- and 230-ft depths is clearly seen in the spread of peak skin frictions recorded at those depths. Although the full-displacement probes tended to obtain values of peak resistance that increased with depth in a manner consistent with the trend shown in the clays, the partial-displacement probe recorded significantly smaller values.

Although there is scatter among the results of the small probe tests, reasonable agreement is also shown among the results of these experiments and those for Pile 2 tested between the depths of 205 and 255 ft, as given in Refs. 6 and 7. The agreement shown among the full-displacement probes and the 14-in. diameter pile also suggests that the larger pile was probably plugged at the time of driving, at least over a significant portion of its length.

Another comparison among the results of the experiments is given in Plate 54, which shows the variation with depth of the residual skin friction recorded during each initial load test. As seen in the figure, the residual skin friction values are more consistent, both among each other, and in comparison with the estimated undrained shear strengths at the site. The values also agree reasonably well with the values of average shear transfer reported in Refs. 6 and 7 for the reduced values of pile-head load which were obtained after a number of loadings to failure. The agreement shown in Plate 54 is thus probably only a coincidence.

Even better agreement among the results of the small-probe experiments is shown in Plate 55 which shows the variation in the values of the cyclic minimum skin friction with depth. Again, considerable scatter is evident in the soils which possess low cohesion. However, consistency shown in the upper clay layer is quite good. The close agreement among the values of cyclic minimum resistance should be expected. The cyclic minimum shear resistance is probably only a function the void ratio and the clay mineralogy. Values of shear resistance greater than the cyclic minimum are temporary, and are influenced by effects from a number of sources other than merely the size, shape, chemical composition, and distance of separation of the particles.

It should be noted that, during the tests on the 14-in. diameter piles, two-way cyclic tests comparable to those reported herein were not performed. Thus, the values of reduced pile capacity reported in Ref. 7 are larger than would be predicted using the values of cyclic minimum resistance from the in situ model tests, as would be expected.

7.3 Effects of Consolidation on Peak Shear Transfer

The dependence of the peak skin friction upon the degree of consolidation is shown in Plates 56, 57 and 58. The values shown in the plates are given in Tables 1 and 2. For the clay layer between the depths of 120 and 160 ft, a consistent linear variation was observed. The development of peak skin friction at the 210-ft depth is apparently independent of the degree of consolidation, as could be expected in cohesionless soil.

Plate 58, which gives the results of the experiments at the 230-ft depth, also shows considerable disagreement between the results of the two experiments which were performed in two dissimilar soils as previously discussed.

The dependence of the peak skin friction upon the radial effective pressure is shown in Plate 59 for the clay layer between the depths of 120 and 160 ft. The upper graph contains the peak skin friction values plotted against the radial effective pressure prior to loading, while the lower graph presents the same data plotted against the effective pressure at the instant of failure. In both cases, the data appear to show a nonlinear relationship with the degree of scatter being reduced if only the results of the initial tests are considered. The values of shear transfer were taken from Table 2. The corresponding values of radial effective pressure are given in Table 3.

Since, during consolidation, the peak shear transfer had exhibited a linear increase with the increasing degree of consolidation while the total radial pressures had concurrently decreased, a nonlinear relationship between the shear transfer and the radial effective pressure should have been expected. Had the total radial pressure remained constant, as in ordinary drained strength tests in the laboratory, simultaneous increases in both the skin friction and the radial effective pressure would have occurred, implying a second linear relationship between the skin friction and the radial effective pressure.

In the absence of any knowledge of the complete state of stress in the soil, in that the stresses on the vertical and circumferential planes are not known, no firm conclusions may be drawn with respect to the relationship between the mean normal effective pressure and the shear transfer.

The repeatability of the data and the consistency of the trends are very encouraging, and suggest that the development of a rational analytical model to explain the behavior and to extrapolate the measured behavior to other sites appears feasible.

The dependence of the peak skin friction on the radial effective pressure at the 210- and 230-ft depths is given in Plate 60. No direct relationship can be established for the soil at the 210-ft depth, with considerable differences shown among the results of the three experiments.

For the soils at the 229.5- and 230-ft depths, there are two different patterns of behavior. At the 230-ft depth, a linear relationship can be established; a pattern consistent with the behavior expected of a cohesionless soil. At the 229.5-ft depth, the behavior is more consistent with that exhibited by the clay in the layer between the depths of 120 and 160 ft.

Since only two points were obtained at the 250-ft depth, a similar figure for this depth is not given.

Perhaps because of the rapidity of loading with respect to the ability of the soil to drain, the behavior observed for all the load tests in the clays appears to be consistent with that of over-consolidated clays under undrained loading conditions; that is, there are no consistent relationships between the undrained strength (in this case, the shear transfer) and the effective confining pressure. An examination of the stress-paths, and of the pressure-time relationships presented in Volume II, shows that, in all instances, the maximum shear transfer occurs at a lower value of radial effective pressure than that which existed at the end of initial consolidation, particularly during the cyclic load tests. Although the two orthogonal pressures, and therefore the mean effective pressure, are not known, the decreased value of the radial effective pressure at failure, as compared with the consolidation pressure, supports this observation.

An examination of the stress-path plots, and of the pressure-time histories contained in Volume II, shows that another consistent pattern was observed during the tests in the clay. A decrease in the radial effective pressure occurred during the initial loading to failure and the subsequent unloading. During the one-way cyclic tension tests, the radial pressures fluctuated, but no permanent effects were recorded.

During the two-way cyclic tests, however, decreases in the radial pressures were observed, which were later recovered during reconsolidation.

The variation in the radial soil pressures during a two-way cyclic test, similar to that given in Plate 12, are shown in Plate 61. In this figure, the simultaneous variations in shear transfer, pore pressure, and radial effective pressure with time are shown during two cycles of the two-way cyclic test on the X-probe at the 160-ft depth. The behavior exhibited by the clay is dilatant, in that the major changes in pressures coincide with the constant-shear portion of the hysteresis loops. During the period of load reversal prior to yield, the pressures tend to return to an equilibrium (although not static) condition. As shown in the figure, the increase in the radial effective pressure during shear has no effect on the magnitude of the shear transfer. Both patterns of response are similar to those exhibited by over-consolidated clays in laboratory shear tests.

7.4 Shear-Displacement (T-Z) Relationships

As was noted during the discussion of the individual experiments, the recorded shear-displacement curves tended to show one common trend: the tendency for the yield displacement to remain almost constant throughout the different series of static and cyclic tests. Although there is not much difference in the diameters of the two probes, a comparison of the curves recorded by the probes at each depth should indicate whether there are clear and consistent trends showing an effect of diameter on the yield displacement.

One such comparison is shown in Plate 62, which contains the results of the initial load tests on all the probes tested at the 141-ft depth. It should be noted that the degree of consolidation for the partial-displacement 3-in. probe is much greater than that for the full-displacement probes.

A comparison of the two X-probe tests with the full-displacement 3-in. probe test, performed at very nearly the same degree of consolidation, would tend to suggest that a dependence on diameter does exist. However, a comparison of the test results obtained at the same depth, with all the probes at similar degrees of consolidation (Plate 63) does not indicate any such dependence, as all curves display very nearly the same slope and yield displacement for all four tests.

A comparison of the tests performed at the 160-ft depth is given in Plate 64. The two full-displacement probes follow very similar loading paths, with differences occurring only near yield. In much the same manner as in the initial load test at the 141-ft depth, the partial-displacement probe follows a much steeper slope and reaches an intermediate value of maximum shear. Again, no clear trend for a dependence on diameter is shown.

Plate 65 shows a similar comparison among the experiments performed at the 210-ft depth. Although there is a considerable difference in the magnitude of the limiting shear transfer, the initial slopes of the three curves are very similar, and again, no consistent trends toward a diameter dependence are present.

Perhaps the only conclusion that can be drawn from these comparisons is that the shear-displacement behavior for the probes in the clay was affected considerably more by the state of consolidation, and thus the stiffness of the adjacent soil, than by diameter differences. The differences in diameter are not large (1.72 vs. 3.00 in.) and thus may not have been sufficiently different for any scale effects to be seen. A more valid comparison could be made between the small-probe test results and the results of the tests on the 14-in. diameter pile during the Chevron pile test program, provided that reliable t-z curves from the earlier tests were available.

7.5 Effects of Rate on Shear Transfer

Near the conclusion of each series of load tests, a study was performed to assess the effects of rate on the limiting shear transfer. In order to exclude any effects of time, consolidation, or other variables which influence the shear transfer, the studies were performed at the end of the two-way cyclic load tests. The initial cyclic loading was performed at a slow rate and consisted of applying several cycles of load reversal until the shear transfer had been reduced to a near-constant value. The rate of travel of the hydraulic ram was then increased to the maximum possible rate, and the two-way cycling continued until the response was again repeatable. Upon conclusion of the fast-rate cycles, at least two additional cycles were then applied at the slow initial rate in order to verify repeatability.

The results of one such series of two-way cyclic tests are given in Plate 66, which were recorded during the X-probe tests at the 120-ft depth. As shown in the figure, the effects of loading rate predominantly affected the value of shear during plastic slip, with the peak shear during the slower loadings being equal to, or only slightly greater than, the shear during the faster loading. The shear transfer during the plastic portion of the test was 0.73 ksf during the slow tests versus 0.79 ksf during the fast cyclic tests. The slip rate varied from 0.0017 in./sec to 0.0321 in./sec during the slow and fast tests, respectively. The increase in shear transfer during these tests was thus about 6 percent per log cycle of increase in the rate of slip, a value which could be reasonably expected for a soft clay.

The results of all the rate-effect studies are included in Volume II, with the results also summarized in this volume in Table 4. Load rate effects were generally consistent in the upper clay layer, with varying effects of rate or the resistance being recorded at the deeper depths. As noted in Plate 44, increases in resistance in the lower soils were much greater than those recorded in the clay, with a maximum recorded rate-sensitivity of 34 percent per log cycle, similar to that reported in Ref. 7 for Pile 2 in the same soil.

7.6 Effects of Diameter and Plug Formation on Consolidation

Because of the unusual consolidation characteristics observed in the soil at the 210-ft depth, a comparison of the effects of diameter during the three experiments will not be made for this particular depth.

As can be seen in Plates 17 and 25, the three full-displacement probes exhibited very similar consolidation behavior. In contrast, the partial-displacement probe consolidated at a faster rate. An examination of the effects of the immediate load test on the consolidation curves (better seen in Volume II) shows that the disturbance, due to the reversed shearing, affected the shape of the curves by various amounts. The modification to the form of the curves indicates that exact correspondence among the results of the experiments is probably not to be expected; however, the curves have enough similarity that a comparison of the behavior will be quite useful.

Since two of the full-displacement probes failed to reach a 50 percent degree of consolidation prior to performing the major series of load tests at the 141-ft depth, the times corresponding to the 40 percent degree of consolidation will be compared. At the 160-ft depth, all the probes reached a degree of consolidation of more than 80 percent; the times corresponding to 40 and 60 percent of consolidation will be used at this depth. This choice of times allows a comparison among the behavior at both depths for all three probe types, and also allows a better comparison of the curve shapes at a wider range of consolidation for the deeper depth.

The rate of consolidation around a pile embedded in clay is a function of the compressibility and permeability of the soil, the diameter of the pile, and the degree of cavity expansion which occurs during installation, that is, the amount of soil which is forced outward during installation.

For one-dimensional consolidation of clay layers of constant thickness, consolidation time is independent of the magnitude of the applied pressure increment (within reasonable limits). For two layers of the same clay having different thicknesses, the time required to attain the same degree of consolidation in each layer is related to the ratio of the squares of the thickness of each layer.

In the case of full-displacement piles, the thickness of the layer of clay which is forced outward during installation, and the thickness of the clay layer which consolidates, should be related to the pile diameter. In the case of the full-displacement probes, which showed similar values of excess total and pore pressure after installation, the times required to attain the same degree of consolidation should, therefore, be in proportion to the squares of the diameters of the probes.

In the case of the full- and partial-displacement probes of equal diameter, the times required to attain the same degree of consolidation are related to the amount of cavity expansion which occurred, that is, the amount of clay which was forced outward during the installation of the probes.

In the experiments at the 141-ft depth, the time required for the two X-probes to reach a degree of consolidation of 40 percent ranged from 125 to 130 minutes. The 3-in. full-displacement probe required approximately 510 minutes to achieve the same degree of consolidation. The ratios of the times are 4.1 and 3.9, respectively; a value which is greater than the factor of 3 which would be expected as the square of the ratio of the diameters of the probes.

In the experiments at the 160-ft depth, the times required for the X-probe to reach degrees of consolidation of 40 and 60 percent were 125 and 430 minutes, respectively. For the 3-in. full-displacement probe, the corresponding times were 405 and 1,230 minutes. The ratios of the times are 3.2 and 2.9, respectively, therefore, in much better agreement with that expected.

The close agreement among the X-probe tests at the two depths indicates that the clay layer was very uniform. At the 120-ft depth, the time required to reach the 40 percent degree of consolidation was 110 minutes, again showing reasonable uniformity. The reasons for the lack of better agreement for the two depths in the 3-in. probe experiments are not readily apparent; perhaps differences in shape, method of installation, or installation time were contributing factors.

For the partial-displacement 3-in. probe at the 141-ft depth, the time required to reach a 40 percent degree of consolidation was only 80 minutes, suggesting that the open-end cutting shoe had, indeed, performed as expected. The ratio of the times for the two 3-in. probes at this degree of consolidation is 6.4 This would tend to indicate that the layer of consolidating clay around the full-displacement probe was perhaps 2.5 times the thickness of that around the partial-displacement probe.

At the 160-ft depth, the times corresponding to degrees of consolidation of 40 and 60 percent were 205 and 720 minutes, respectively. Again the partial-displacement probe had consolidated at a faster rate, with the ratios of the times being 2.0 and 1.7, respectively, as compared with the full-displacement probe of the same diameter. The corresponding layer thickness ratios would be inferred to be 1.4 and 1.3, again indicating that less clay was affected by the insertion of the partial-displacement probe.

In addition to the variation already noted in the shear transfer behavior of the soil at the lower depths, a comparison of the consolidation behavior during the X-probe experiments may be used as a measure of the nonuniformity of the soil. The variation in the times required for an equal degree of consolidation at each depth, all else being equal, is a measure of the variation in the consolidation coefficient, c_v. As already noted, the repeatability among the several X-probe experiments in the clay layer between the depths of 120 and 160 ft was excellent; similar repeatability would also be expected for a uniform soil at the deeper depths.

Choosing a common degree of consolidation of 50 percent for all depths below 160 ft, a comparison of the variations in the time required for consolidation shows the following: At the 160-ft depth, the time was 234 minutes; at the

210-ft depth, the time was 21 minutes; at the 229.5 and the 230-ft depths, the times were 76 and 214 minutes, respectively; and at the 250-ft depth, the time was 400 minutes.

The range in the consolidation times for the experiments at the three depths suggests a wide variation in the consolidation coefficient, c_v. Only at the 250-ft depth is the behavior indicative of the soil type shown in Plate 2 for this layer. At the 230-ft depth, the clay was similar to that in the upper layer, in terms of consolidation behavior. At the 250-ft depth, the consolidation time is greater than that in the upper layer. The consolidation coefficient, c_v, generally decreases with the liquid limit. Since the time required for consolidation is inversely proportional to the value of c_v, the lower liquid limits shown in Plate 2 would indicate that a longer consolidation time would be expected for the clay at this depth.

8.0 CONCLUSIONS AND RECOMMENDATIONS

The results of our program of research at the Empire pile load test site have shown that the two in situ tools developed at The Earth Technology Corporation are quite useful in obtaining the type and quality of data required to advance the knowledge of the mechanics of axial pile-soil interaction, and can also be used to obtain pile design parameters directly. Although a comparison of the deformational response among the several experiments was inconclusive, the data which was obtained may be further compared with the results of the large pile tests to determine the effects of diameter on the shear-displacement behavior.

Among the questions which were answered by the experiments are:

1. In the clay layers, pile plugging has little effect on the maximum shear transfer, and therefore, pile capacity.

2. In layers of rapidly consolidating soils possessing low cohesion, the degree of cavity expansion and pile plugging have significant effects on the frictional capacity.

3. The effects of diameter on consolidation rate were approximately those expected; i.e., the consolidation time is roughly proportional to the square of the pile diameter, given equivalent boundary conditions, i.e., plugged or unplugged.

4. The effects of plugging on consolidation were also reasonable, being related to the degree of cavity expansion, and thereby, the thickness of the consolidating layer. If the amount of cavity expansion can be realistically modelled, or predicted, then the process of consolidation can also be reasonably well predicted.

5. The linear increase in shear transfer with degree of consolidation agreed well with the behavior observed for clay soils in traditional laboratory strength tests.

6. The decreases in the radial total pressure during consolidation led to a nonlinear relationship between the maximum shear transfer and the radial effective pressure. Had the components of effective pressure in the two orthogonal directions been known, perhaps the relationship would also have been linear with respect to the mean effective pressure, as is experienced in normal laboratory strength tests.

7. Based solely on the variations in the radial effective pressure during the load tests, the behavior of the clay near an axially-loaded pile most resembles that of an over-consolidated clay, when compared to the behavior observed in ordinary laboratory tests. Because of the radial variation in void ratio and shear strength after reconsolidation, such behavior should not be surprising.

8. One-way cyclic tension loading had no effect on the maximum shear transfer; the effects on the deformational response (t-z) were also of little consequence.

9. Two-way cyclic loading had varying degrees of effect, with losses in the shearing resistance ranging from 25 to 40 percent while the yield displacement remained fairly constant throughout the degradation process.

The results of the experiments at Empire indicate that the traditional methods used in the design of axial piles in normally consolidated clays, in terms of ultimate capacity, are reasonably sound, provided that sufficient time is allowed for consolidation prior to the occurrence of the design loading.

The trends in behavior recorded during the experiments in the clay are very encouraging, in that the consistency suggests that the development of a method for prediction of the development of shear transfer with time, including the effects of shear and cyclic loading on pile capacity, is an attainable goal.

For the design of large-diameter piles, particularly those used to provide the foundations for the anchoring of tension leg platforms, the development of a method which can, with reasonable confidence, predict the time-history of the capacity of the foundation piles is sorely needed. For such piles, which may have diameters of 84 in. and larger, the economics of construction, and production schedules, require that the foundation support the design loading quite some time before the soil has reached an advanced degree of consolidation.

The traditional concepts of factors of safety based on a fully-consolidated ultimate pile capacity are of little value for such foundations. In order to estimate the actual capacity of the foundation piles at the time the service loads, and perhaps the design storm loads, are expected to occur, a method must be developed to predict the development of pile capacity with time. The method must realistically assess the degree of cavity expansion which is created during pile installation, so that the modelling of consolidation time does not result in undue conservatism in the design.

The results of the open-end 3-in. probe experiments indicate that the use of published methods for predicting consolidation time, which disregard the effects of the degree of cavity expansion, tend to overpredict the consolidation time to varying degrees. Such overprediction of the time required to develop the capacity of the foundation of an actual structure can have a serious impact on the economic, and perhaps even the structural, feasibility of the design.

The results of the experiments in the soil at the 210-ft depth suggest that much remains to be learned about the mechanics of axial pile behavior in cohesionless soils. The soil at the 210-ft depth was probably a silt, however, in the absence of actual samples, the soil type is not accurately known. The results of the experiments indicate that the capacity of a pile foundation in such soils would depend, to a great extent, upon the method used to install the piles, with the effects of installation changing the capacity by a factor of at least two.

9.0 REFERENCES

1. Vijayvergia, V. N., and J. A. Focht, Jr., "A New Way to Predict the Capacity of Piles in Clay", Proceedings, 4th Offshore Technology Conference, Houston, Texas, 1972, Vol. 2., pp. 865-874.

2. Dennis, N. D., and R. E. Olson, "Axial Capacity of Steel Pipe Piles in Clay", Proceedings, Geotechnical Practice in Offshore Engineering, ASCE, Austin, Texas, 1983, pp. 370-388.

3. Kraft, L. M. Jr., "Effective Stress Capacity Model for Piles in Clay", Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, No. GT11, 1982, pp. 1387-1404.

4. Kirby, R. C,. M. E. Esrig, and B. S. Murphy, "General Effective Stress Method for Piles in Clay", Proceedings, Geotechnical Practice in Offshore Engineering", ASCE, Austin, Texas, 1983, pp. 457-498.

5. Pelletier, J. H., and E. H. Doyle, "Tension Capacity in Silty Clays", Proceedings, 2nd International Conference on Numerical Methods in Offshore Piling, Austin, Texas, 1982, pp. 163-181.

6. Cox, W. R., L. M. Kraft Jr., and E. A. Verner, "Axial Load Tests on 14-Inch Pipe Piles in Clay", Proceedings, 11th Offshore Technology Conference, Houston, Texas, Vol. 2, 1979, pp. 1147-1158.

7. Kraft, L. M. Jr., Cox, W. R., and Verner, E. A., "Pile Load Tests: Cyclic Load and Varying Load Rates", Journal of the Geotechnical Division, ASCE, Vol. 107, No. GT1, 1981, pp. 1-19.

8. Matlock, H., I. P. Lam, and L. C. C. Cheang, "Analytical Interpretation of Pile Installation and Axial Performance", Proceedings, 2nd International Conference on Numerical Methods in Offshore Piling, Austin, Texas, 1982, pp. 133-162.

TABLES

TABLE 1. SUMMARY OF MEASURED PORE PRESSURES

Depth	Probe	u_{max}	u_o	(a) u_{test}	U,%	(b) u_{test}	U,%	(c) u_{test}	U,%
120	X	18.32	8.49	16.78	16	10.19	83	9.57	89
141	X(1)	21.85	9.82	19.42	20	15.55	52	12.69	76
	X(2)	21.88	9.82	20.48	11	15.87	47	12.34	75
	3C	20.70	9.82	20.11	5	15.52		12.51	75
	3O	18.12	9.82	16.68	17	10.71	89	10.82	88
160	X	26.46	11.03	24.29	14	13.55	84	12.47	91
	3C	26.85	11.03	25.43	9	13.75	83	12.45	91
	3O	22.26	11.03	17.66	41	11.99	91	—	—
210	X	28.43	14.21	21.71	47	14.10	100	14.21	100
	3C	28.80	14.21	15.92	88	14.23	100	—	—
	3O	21.48	14.21	16.68	66	14.39	100	14.36	100
230	X(1)	34.0*	14.59	33.20	9	23.54	56	16.78	89
	X(2)	33.65	14.59	29.17	24	18.34	80	14.59	100
250	X	37.82	15.87	35.14	12	20.86	77	—	—

Legend:

X = X-probe
3C = 3-in. tool, closed-end shoe
3O = 3-in. tool, open-end shoe
(a) = Immediate test
(b) = Test after undisturbed consolidation
(c) = Test after additional period of undisturbed consolidation following major load test series
* = Estimated

TABLE 2. SUMMARY OF MEASURED SKIN FRICTION

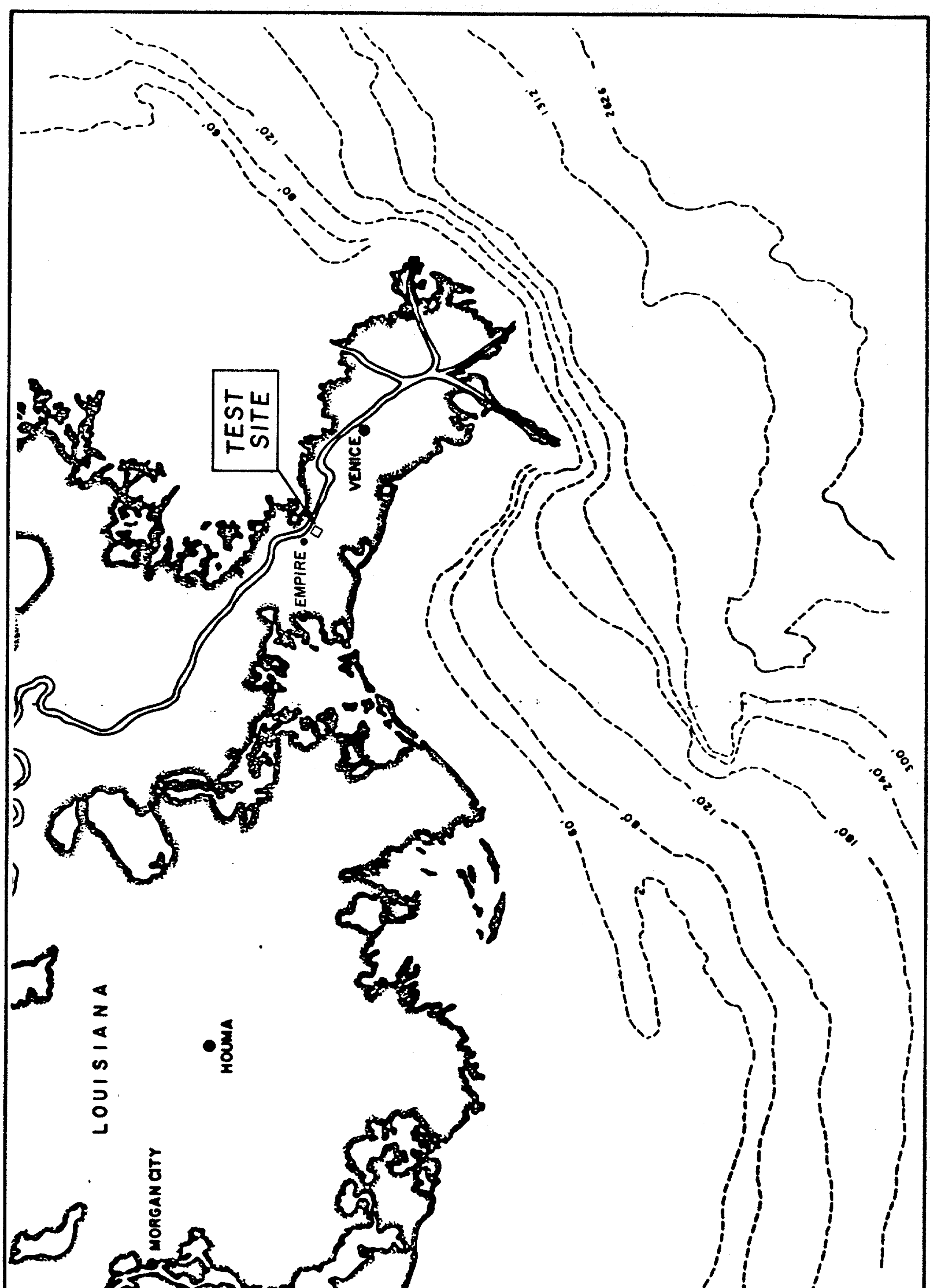
Depth	Probe	Immediate f_{max}	Immediate f_{res}	After Consolidation f_{max}	After Consolidation f_{res}	After One-Way f_{max}	Two-Way Cycling f_{min}	After Further Consolidation f_{max}	After Further Consolidation f_{res}
120	X	0.42	0.36	0.96	0.96	0.97	0.71	1.26	0.89
141	X(1)	-	0.37	0.76	0.76	0.97	0.70	1.28	0.96
	X(2)	0.46	0.38	0.84	0.76	—	0.60	1.11	0.95
	3C	0.38	0.29	0.76	0.66	0.76	0.77*	1.21*	—
	3O	0.59	0.42	1.04	0.90	—	0.78	—	—
160	X	0.40	0.36	1.18	1.18	1.17	0.93	1.41	1.12
	3C	0.46	0.25	1.48	1.22	1.49	0.89	1.61	1.43
	3O	0.48	0.34	1.35	1.16	1.38	0.92	0.97	0.92
210	X	2.03	—	1.94	—	1.80	1.49	1.90	—
	3C	0.85	—	1.78	—	—	1.25	—	—
	3O	0.16	—	0.95	0.63	—	0.69	0.74	0.71
230	X(1)	1.70	—	2.06	—	—	1.75	1.99	—
	X(2)	0.76	—	2.35	-	—	—	2.88	—
250	X	0.66	0.50	1.76	1.72	1.73	1.12	—	—

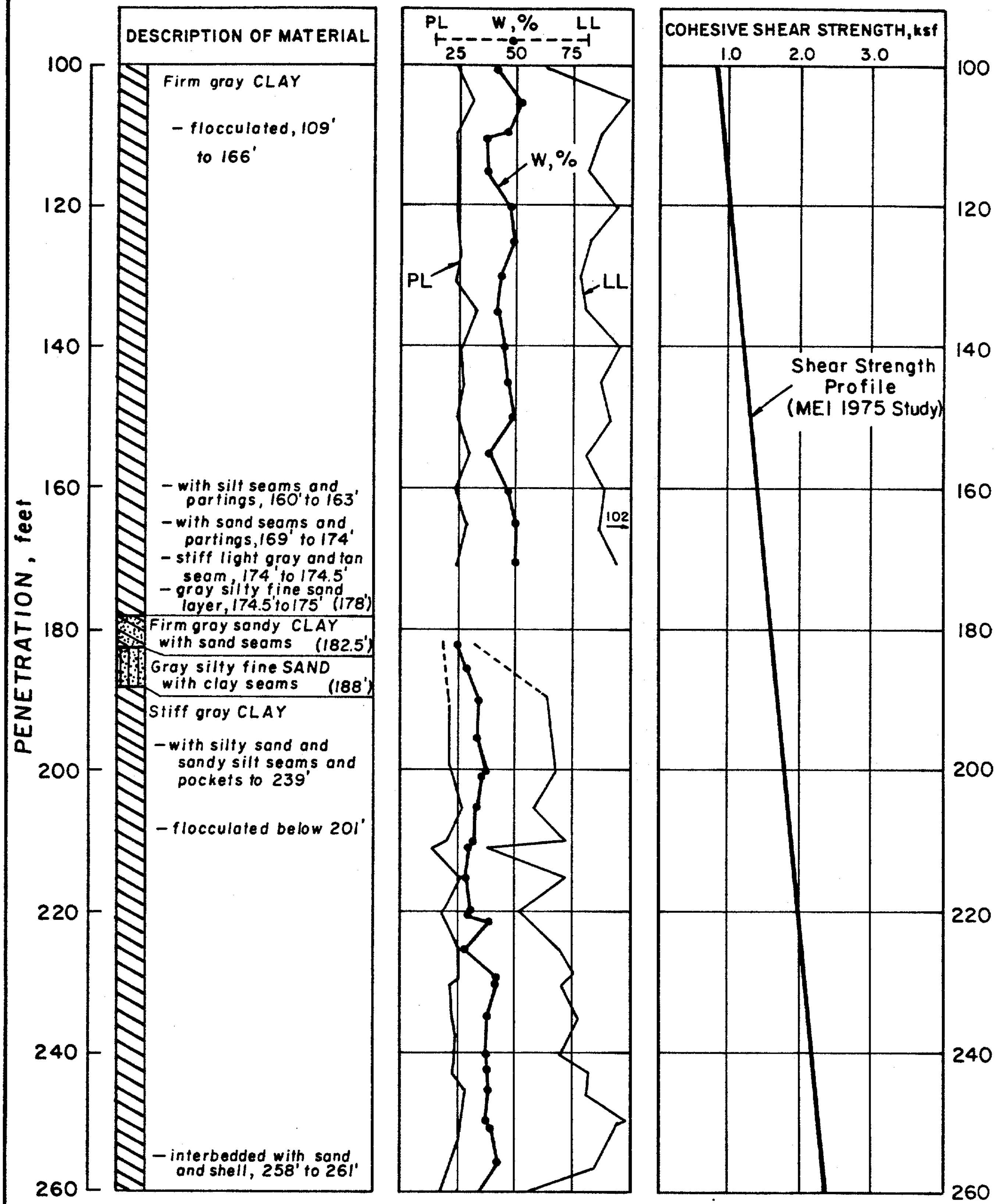
*Values obtained after additional consolidation

TABLE 3. SUMMARY OF RADIAL EFFECTIVE PRESSURES

Depth	Probe	After Consol σ'c	After Consol σ' failure	After One-Way σ' initial	After One-Way σ' failure	Two-Way Cycling σ'f	After Further Consolidation σ'c	After Further Consolidation σ'f
120	X	5.38	4.81	4.56	5.41	3.10	5.06	4.48
141	X(1)	4.69	4.20	4.72	4.87	3.28	5.77	5.14
	X(2)	4.55	4.10	—	—	2.68	6.11	5.55
	3C	5.02	3.10	3.67	3.36	4.23	7.28	5.50
	3O	4.56	4.35	—	—	3.15	3.69	3.66
160	X	7.57	7.14	6.47	7.62	5.10	7.17	6.57
	3C	10.40	7.04	8.52	7.47	5.43	10.13	7.20
	3O	4.84	4.22	3.94	4.04	2.75	—	—
210	X	10.08	9.52	9.11	10.26	6.70	7.92	7.61
	3C	5.86	4.10	—	—	3.56	—	—
	3O	3.17	2.29	2.10	1.90	1.19	1.89	1.52
230	X(1)	6.89	5.93	—	—	5.31	10.65	10.34
	X(2)	12.74	11.13	—	—	—	13.88	14.76
250	X	14.94	13.72	12.33	13.12	10.87	—	—

JOB NO.________

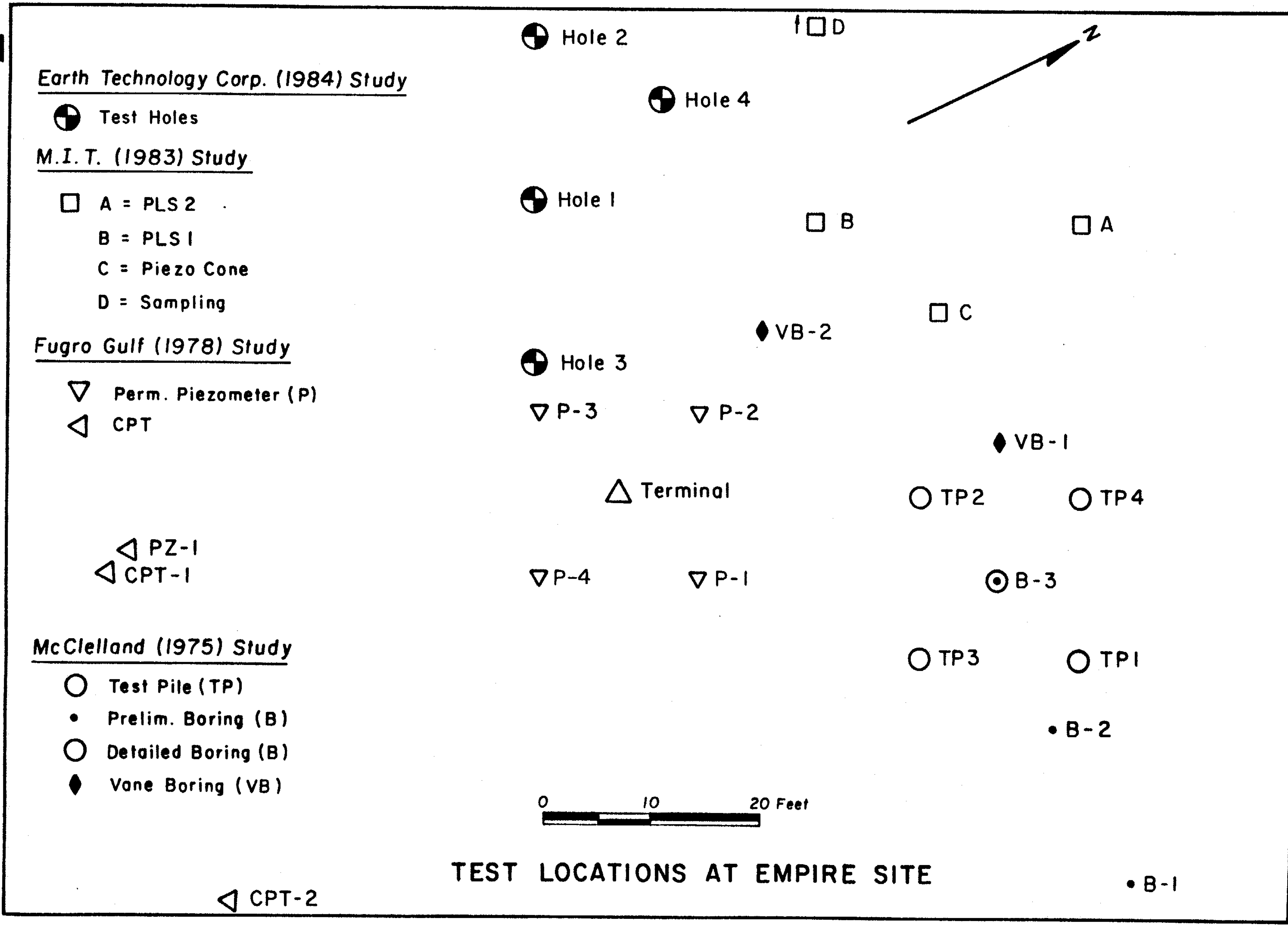

TEST SITE
VENICE
EMPIRE
LOUISIANA
HOUMA
MORGAN CITY
60'
120'
80'
1312'
2626'
60'
80'
120'
180'
240'
300'



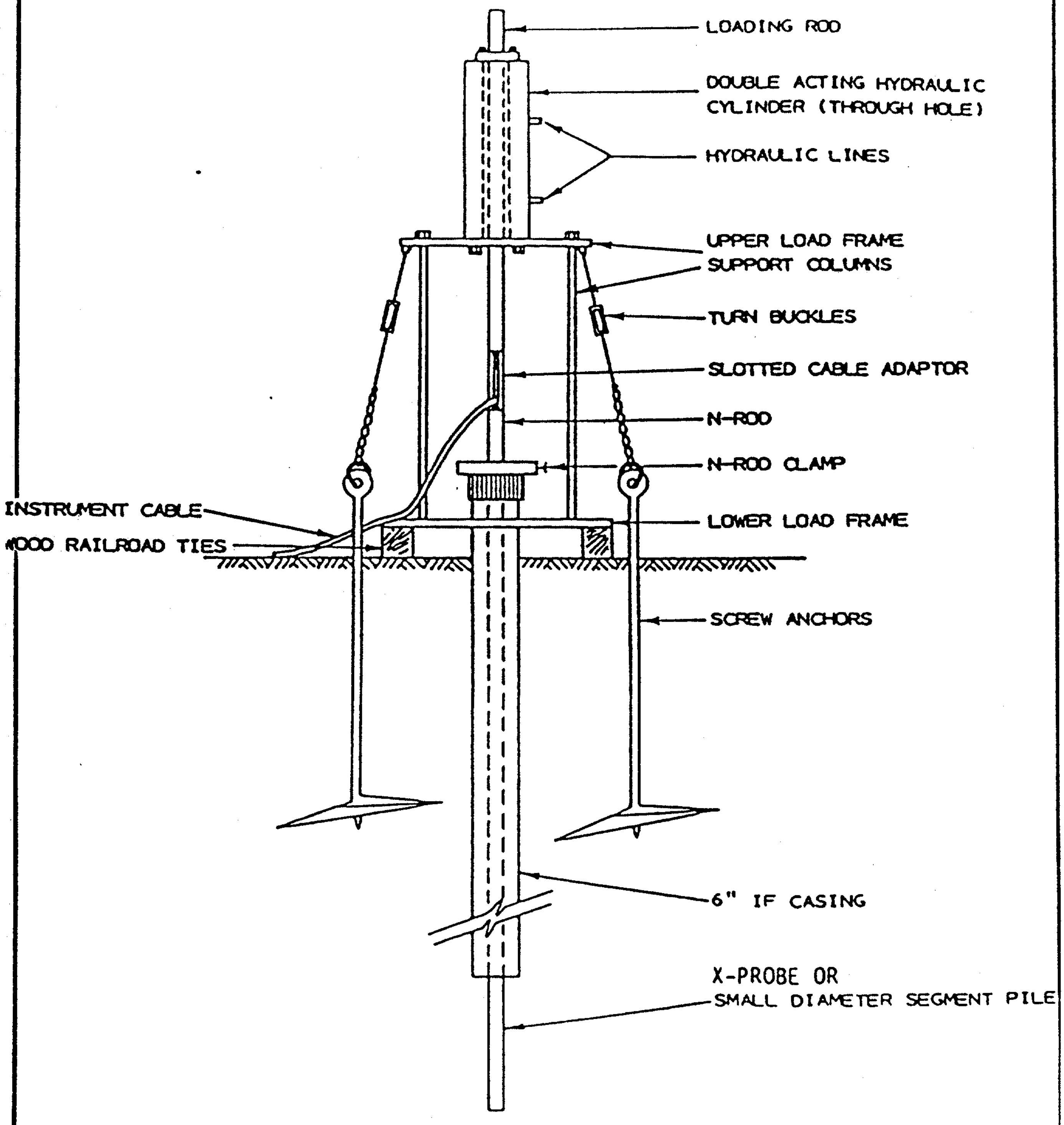
SOIL CONDITIONS AT EMPIRE TEST SITE

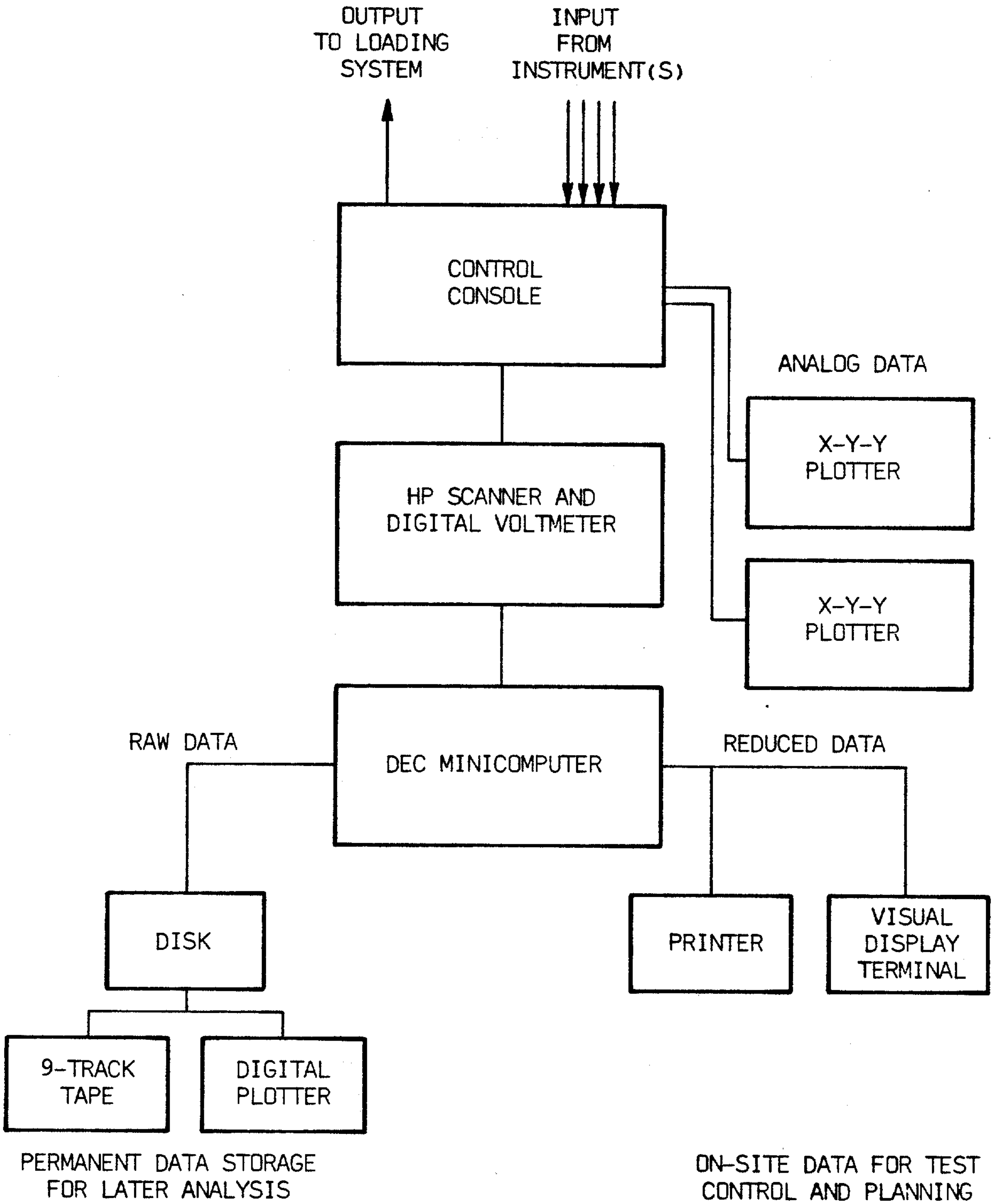
The Earth Technology Corporation

JOB NO.



TEST LOCATIONS AT EMPIRE SITE


LOADING ROD
DOUBLE ACTING HYDRAULIC CYLINDER (THROUGH HOLE)
HYDRAULIC LINES
UPPER LOAD FRAME
SUPPORT COLUMNS
TURN BUCKLES
SLOTTED CABLE ADAPTOR
N-ROD
N-ROD CLAMP
INSTRUMENT CABLE
LOWER LOAD FRAME
WOOD RAILROAD TIES
SCREW ANCHORS
6" IF CASING
X-PROBE OR
SMALL DIAMETER SEGMENT PILE



SCHEMATIC DIAGRAM OF
DATA ACQUISITION SYSTEM
FOR SMALL-DIAMETER PILE SEGMENT TESTS

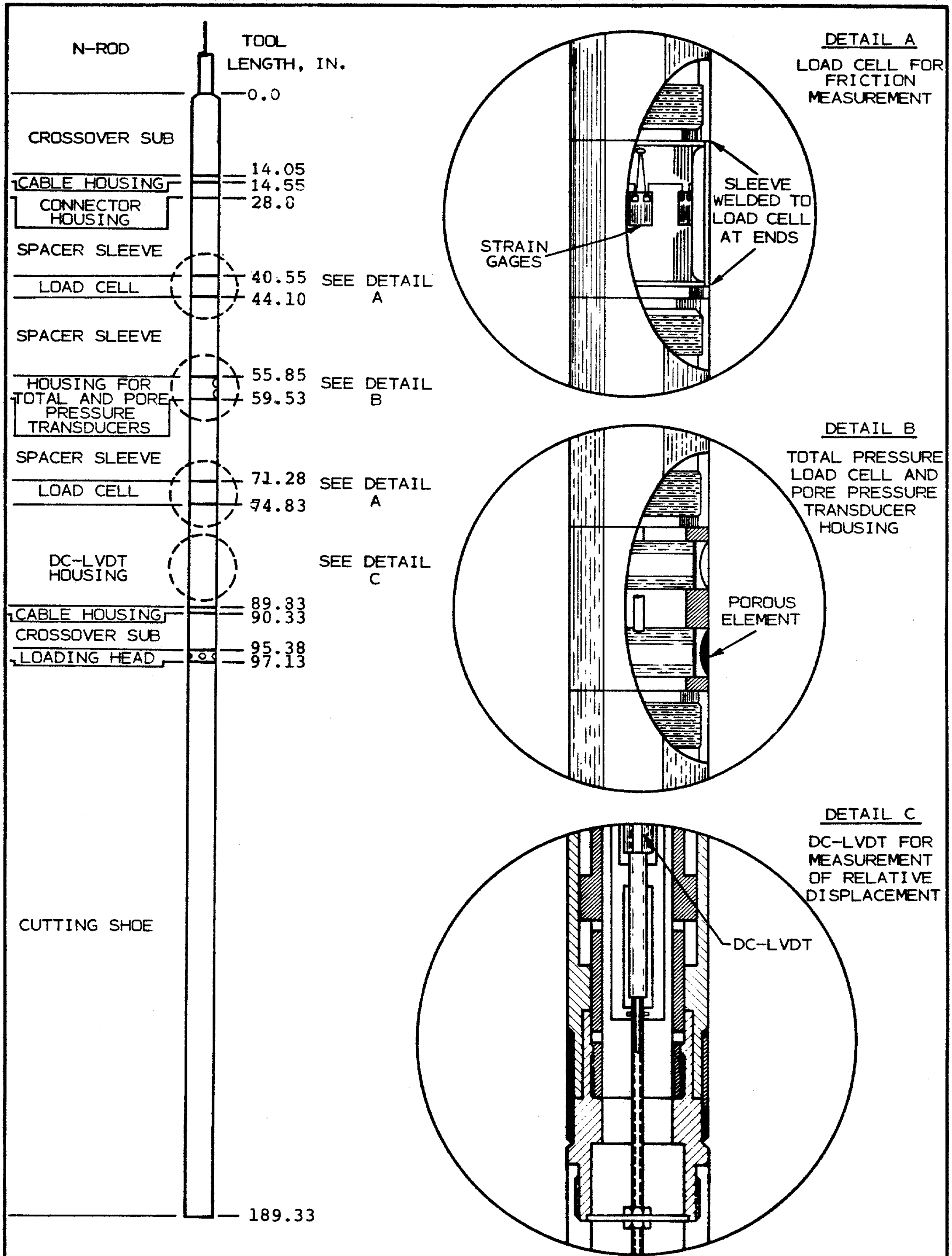
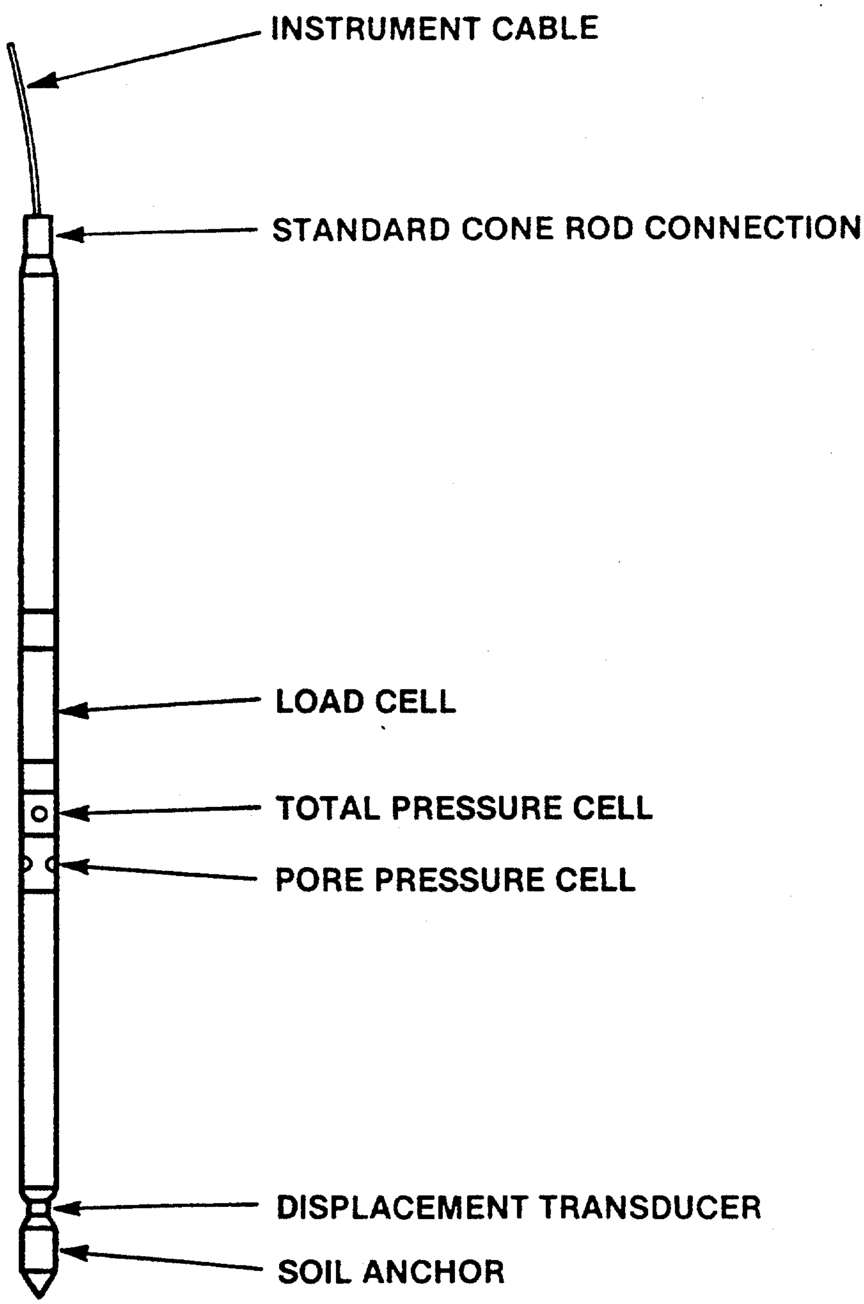


DIAGRAM OF SMALL DIAMETER PILE SEGMENT

JOB NO.


INSTRUMENT CABLE
STANDARD CONE ROD CONNECTION
LOAD CELL
TOTAL PRESSURE CELL
PORE PRESSURE CELL
DISPLACEMENT TRANSDUCER
SOIL ANCHOR

X-PROBE

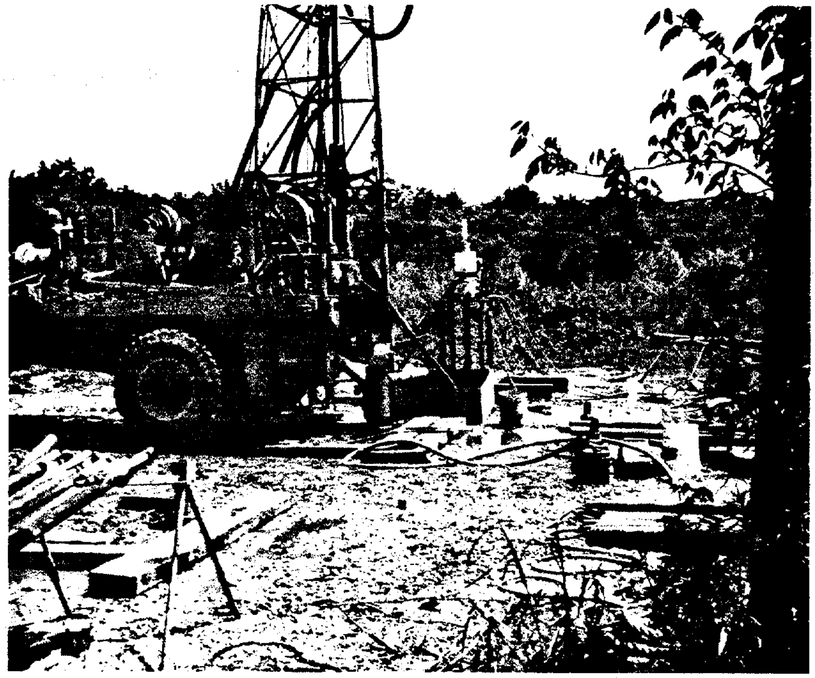


Plate 8. Test Site During Field Operations

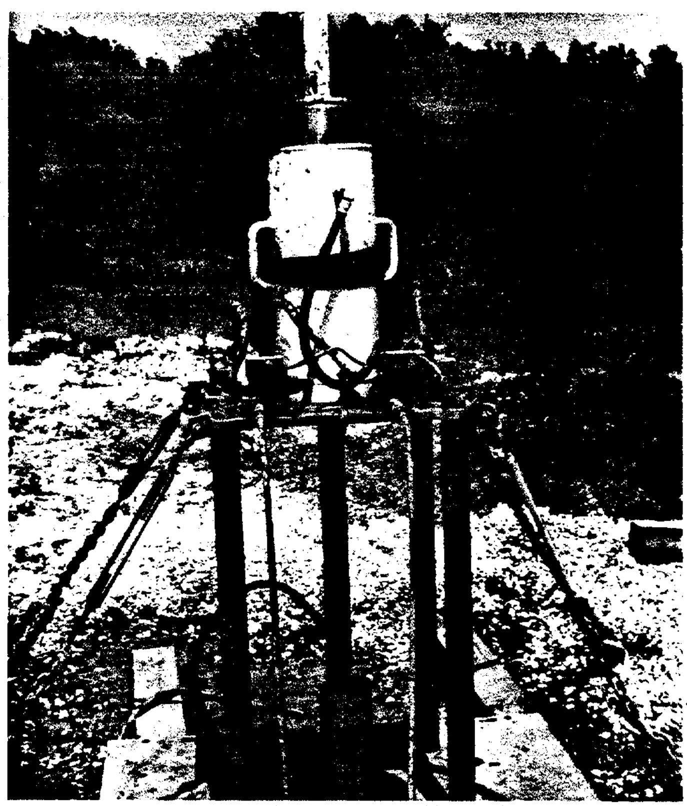
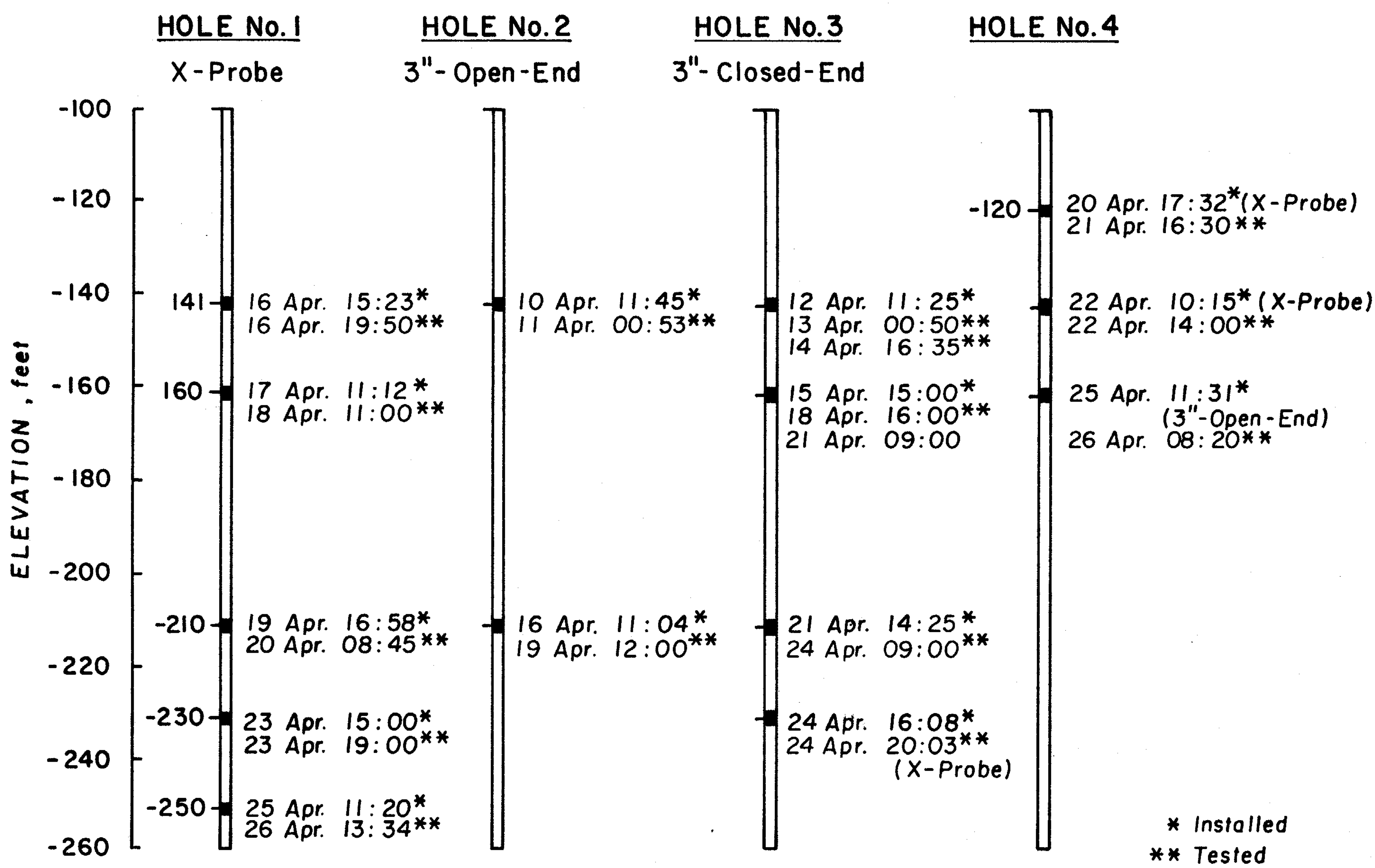
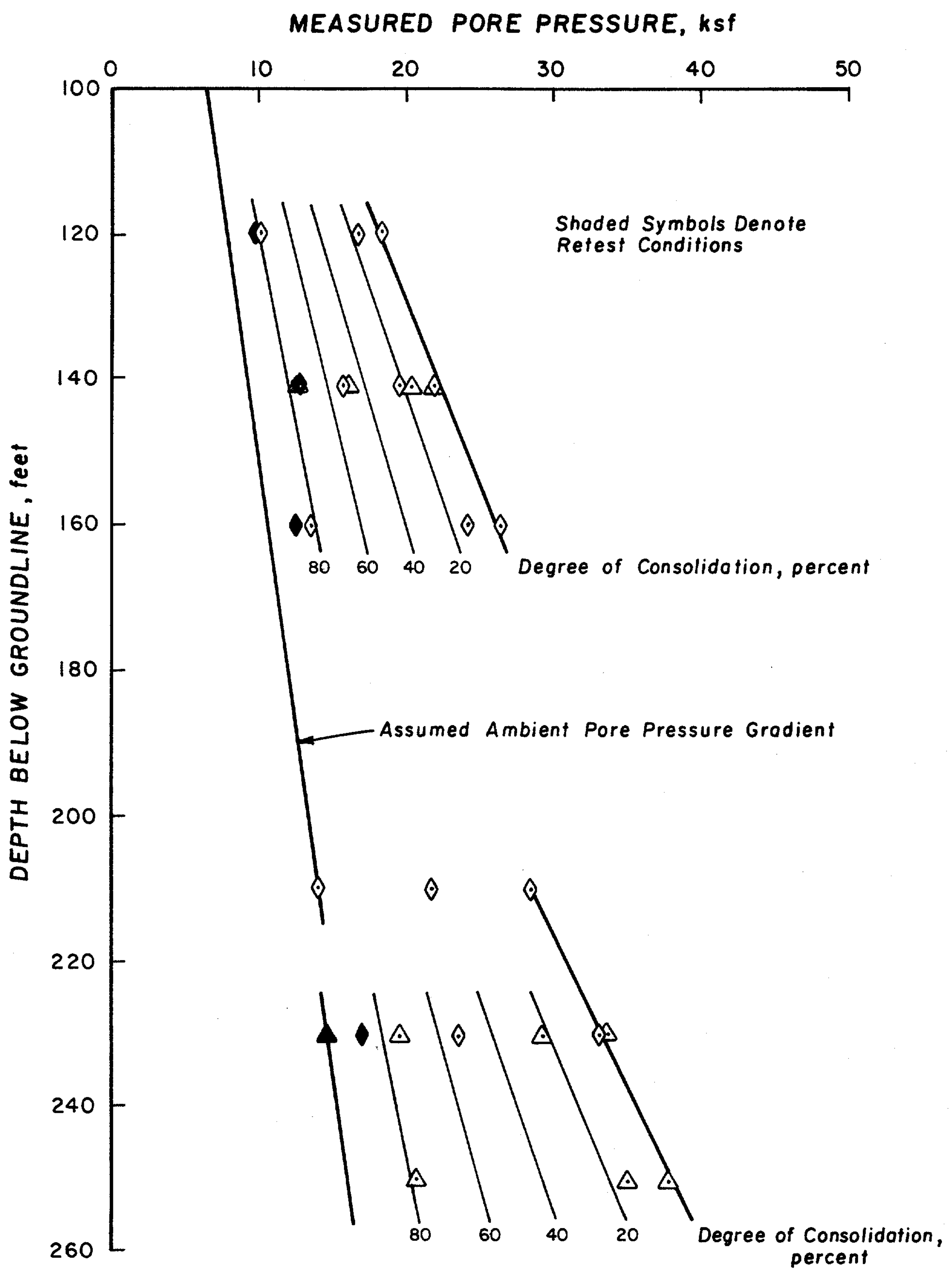


Plate 9. Loading Apparatus in Testing Position

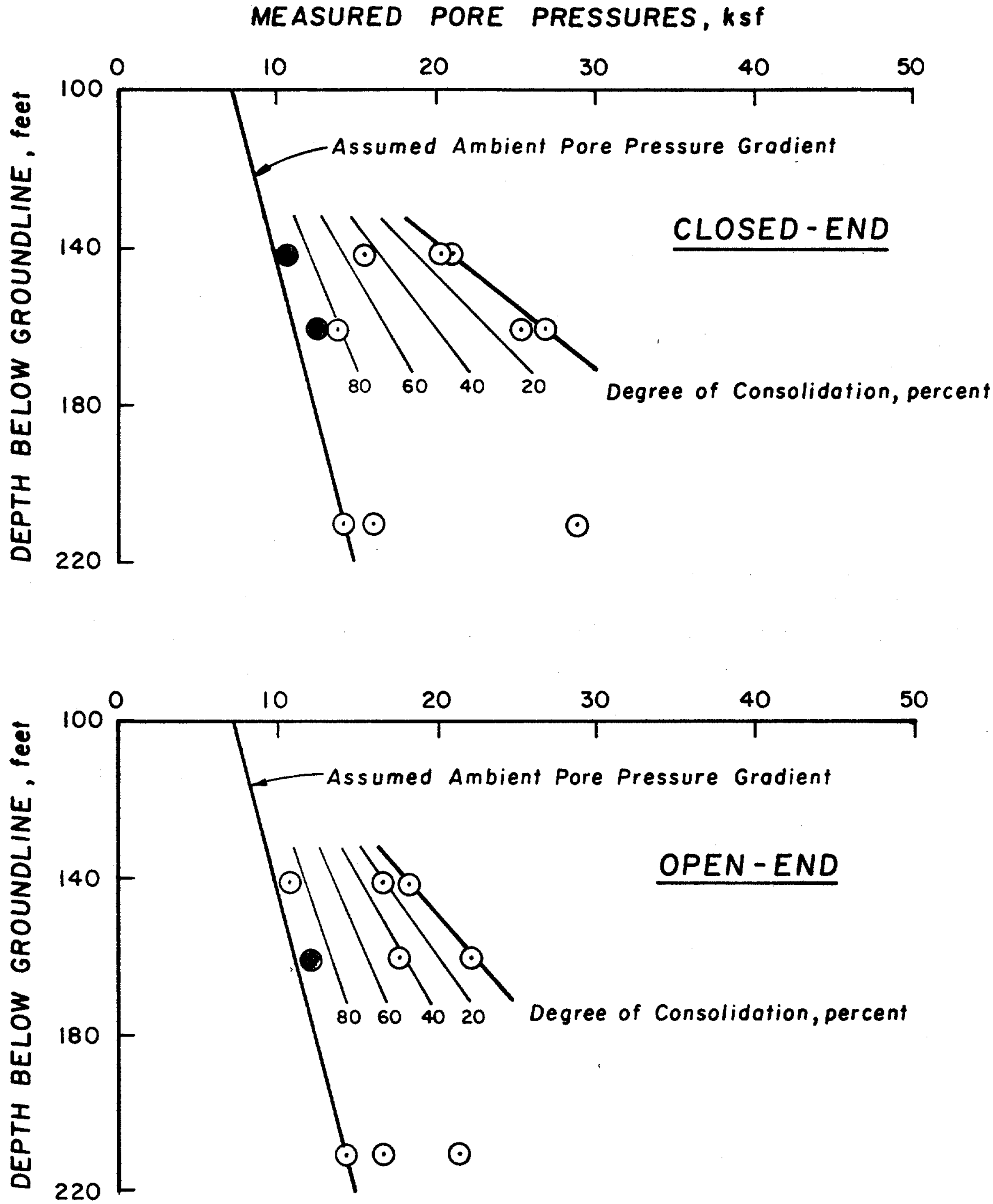


SUMMARY OF SMALL-DIAMETER PILE SEGMENT TESTS AT EMPIRE, LOUISIANA

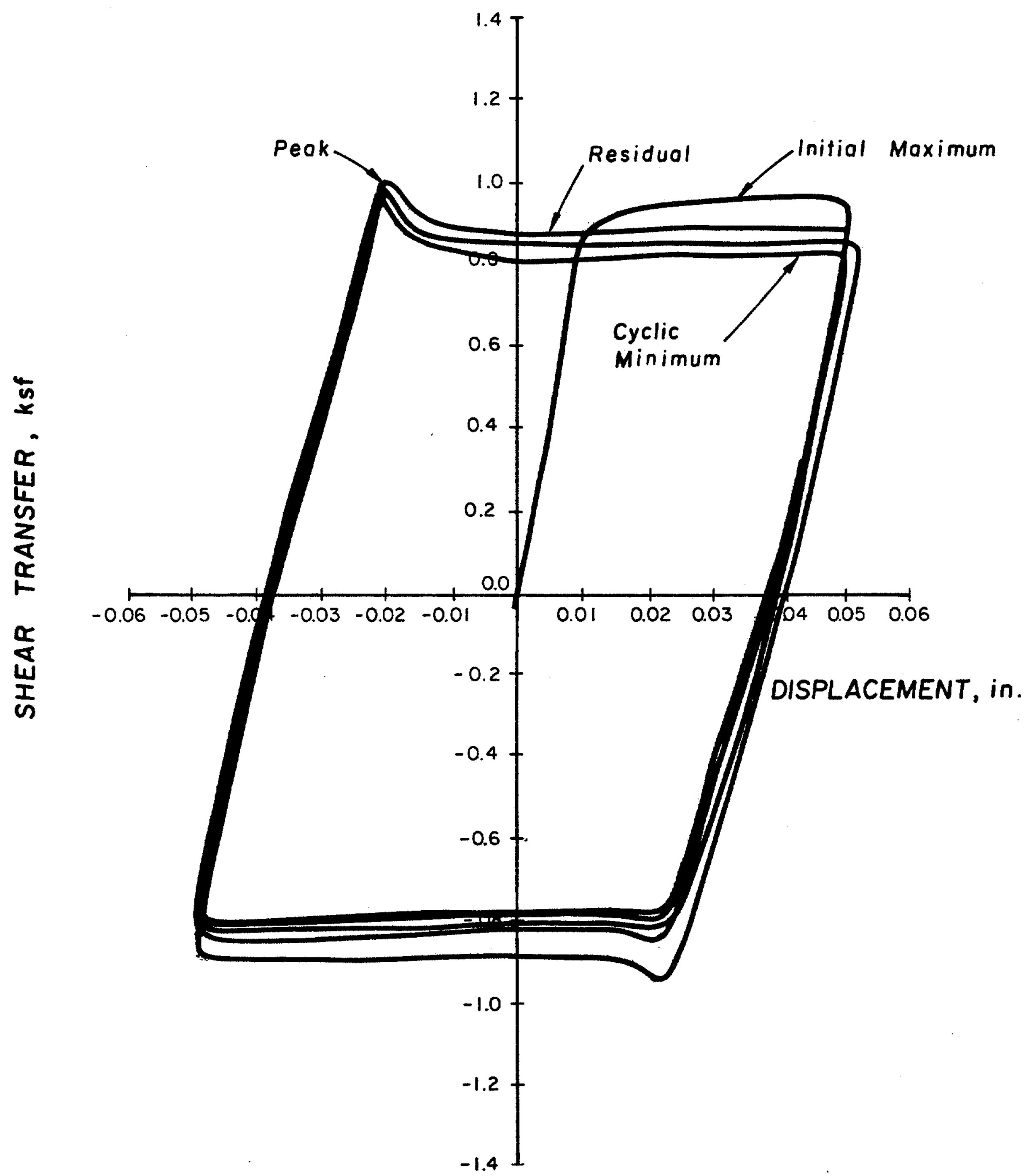
The Earth Technology Corporation



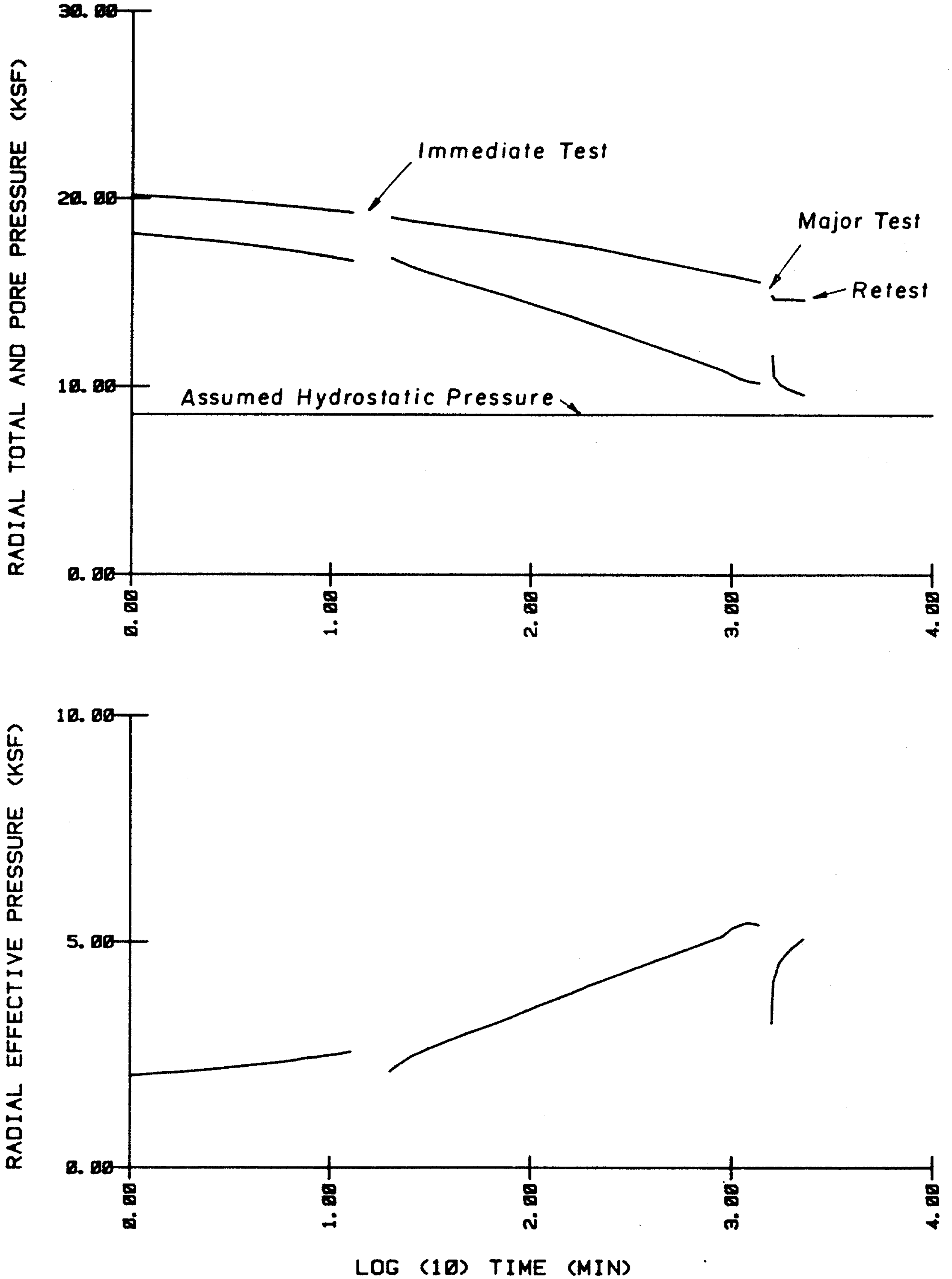
SUMMARY OF X-PROBE EXPERIMENTS



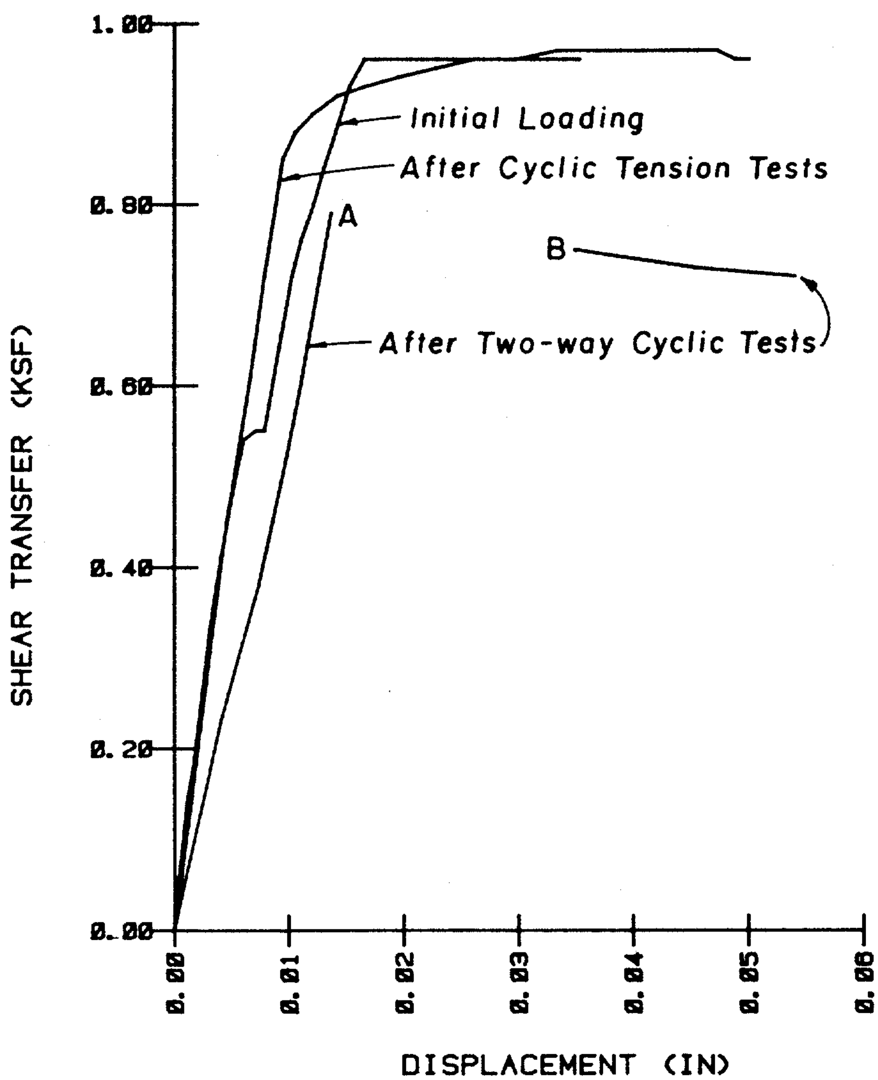
SUMMARY OF 3-inch DIAMETER PROBE EXPERIMENTS



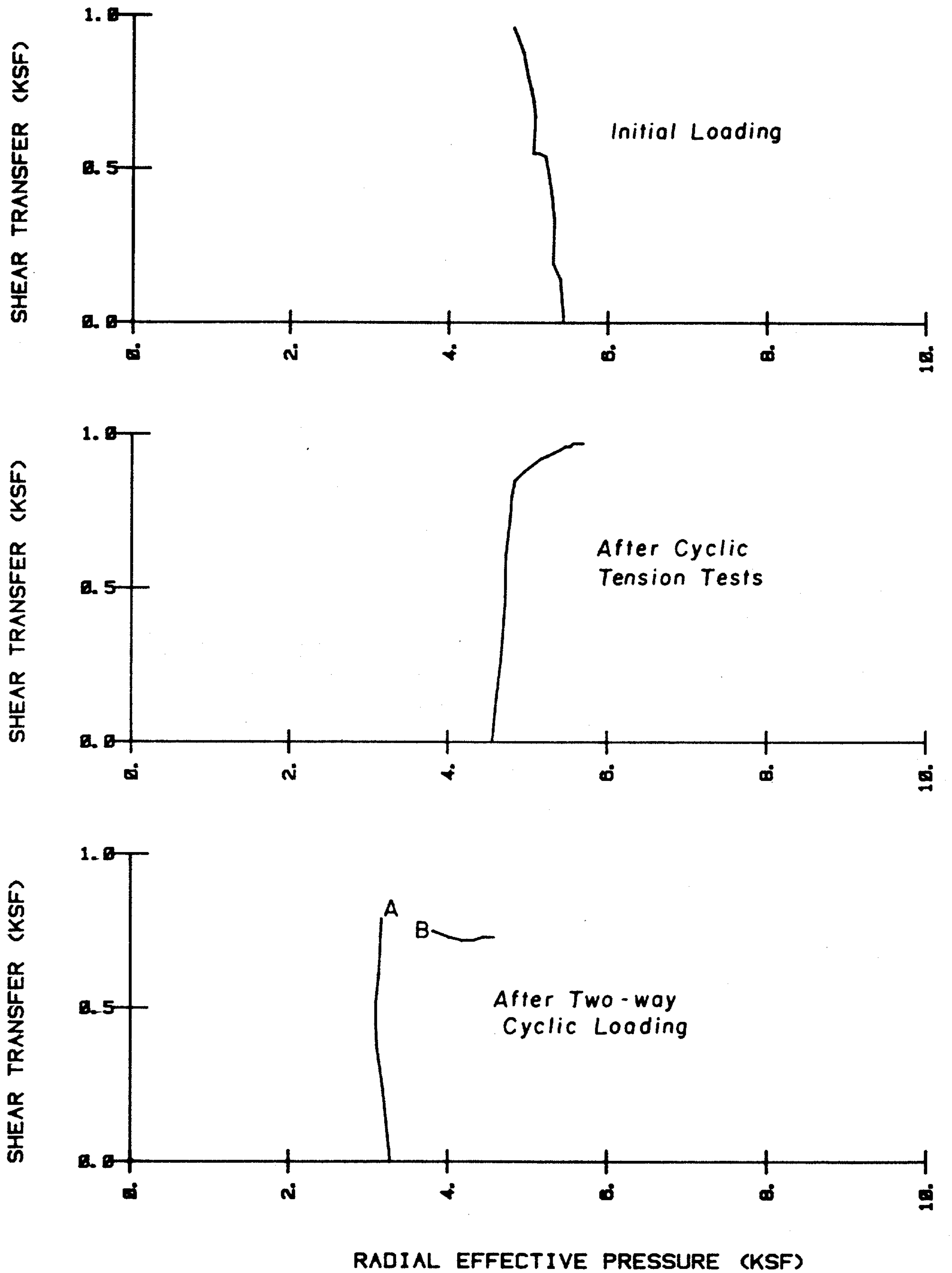
RESULTS OF THE TWO-WAY CYCLIC TEST
AT THE 120-FT DEPTH (X-PROBE)



SOIL PRESSURES DURING CONSOLIDATION AT THE 120-FT DEPTH
(X-PROBE)



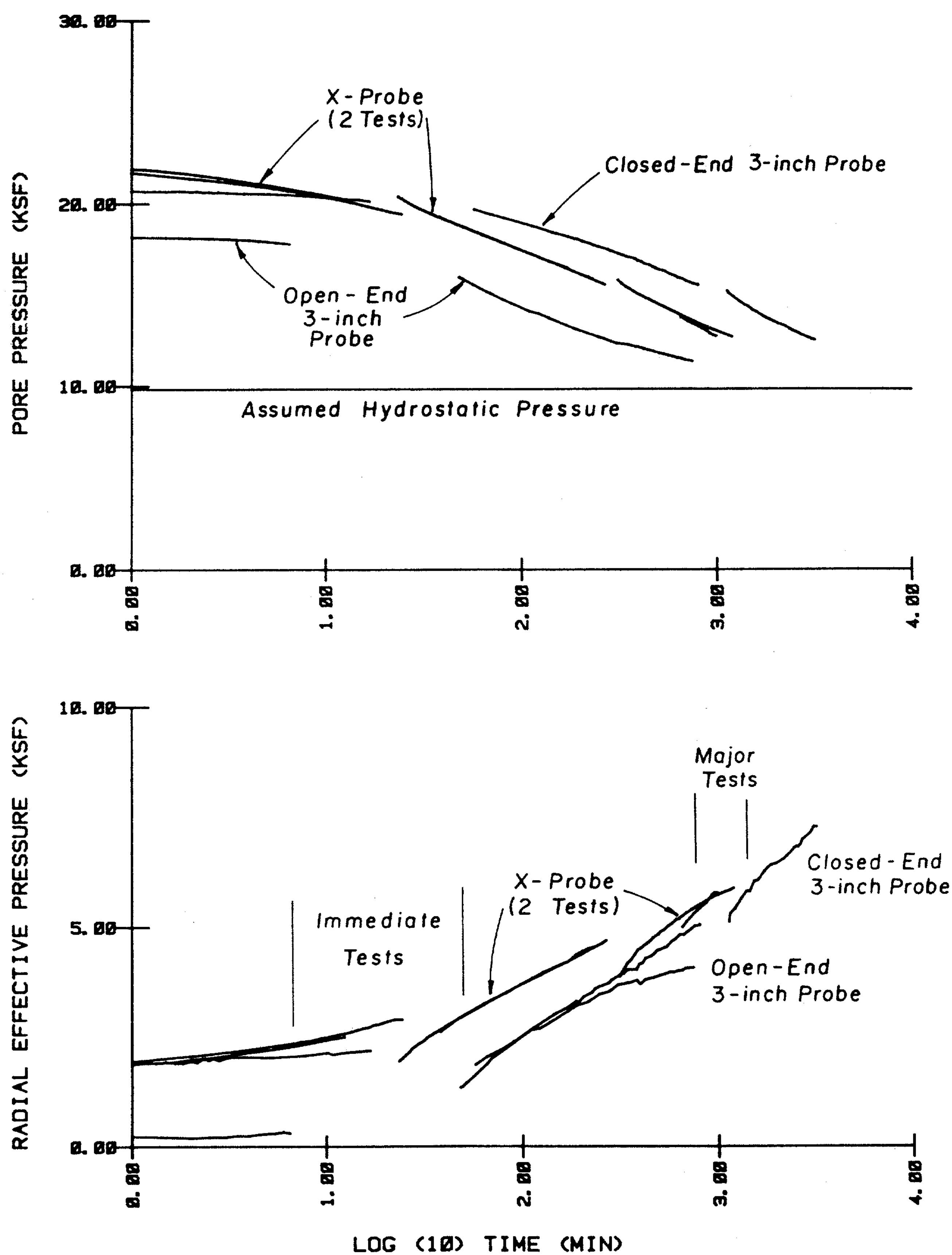
RESULTS OF THE CYCLIC TEST PROGRAM AT THE 120-FT DEPTH
(X-PROBE)



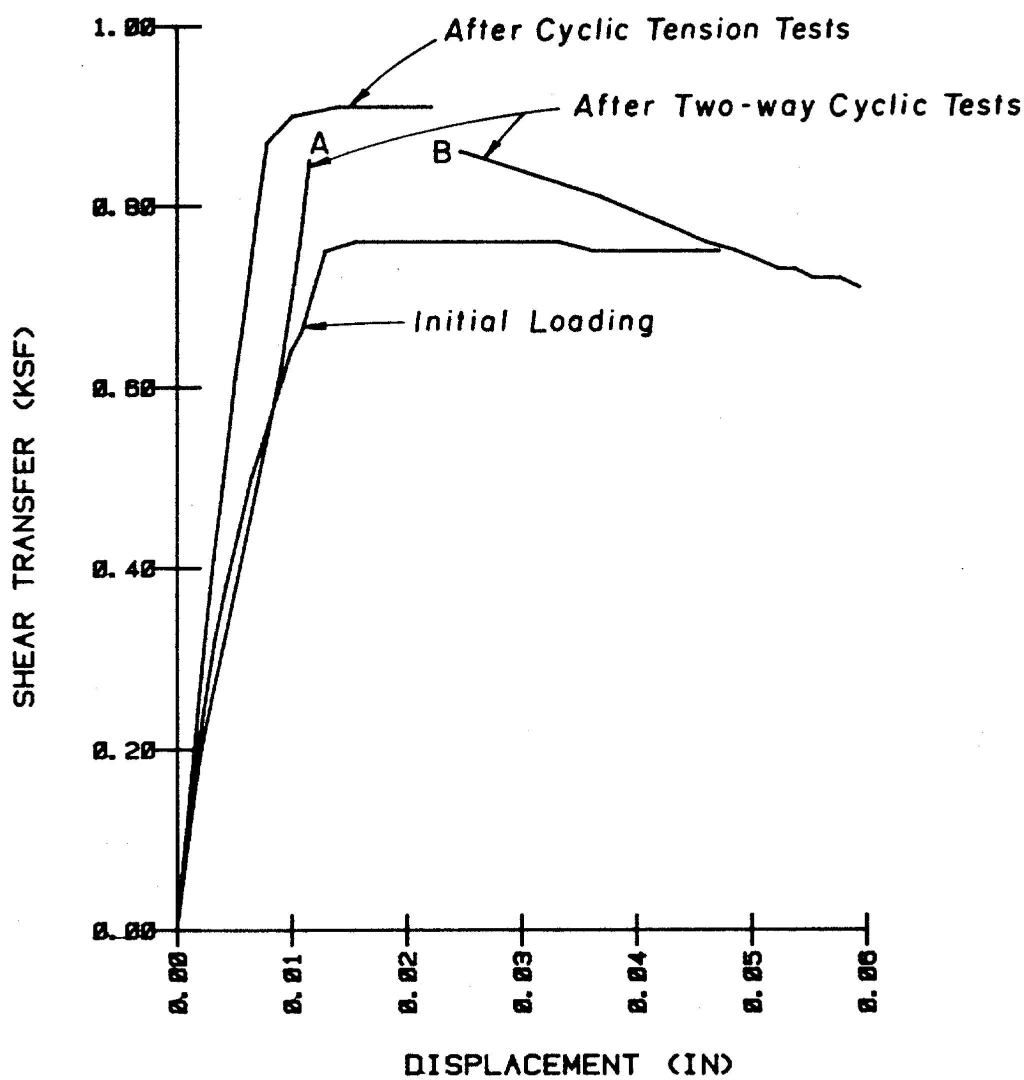
STRESS PATHS DURING LOAD TESTS AT THE 120-FT DEPTH
(X-PROBE)

The Earth Technology Corporation

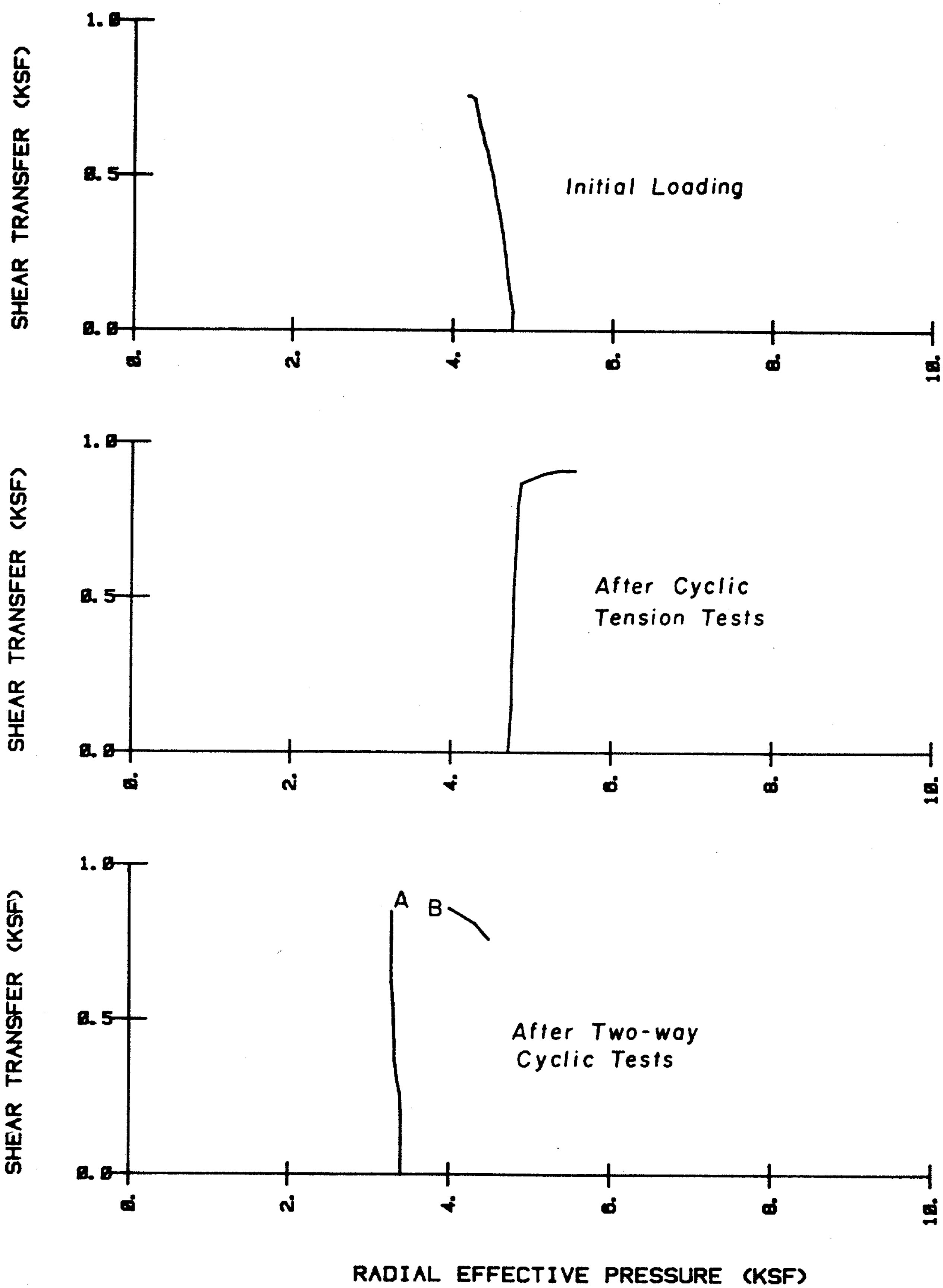
PLATE 16



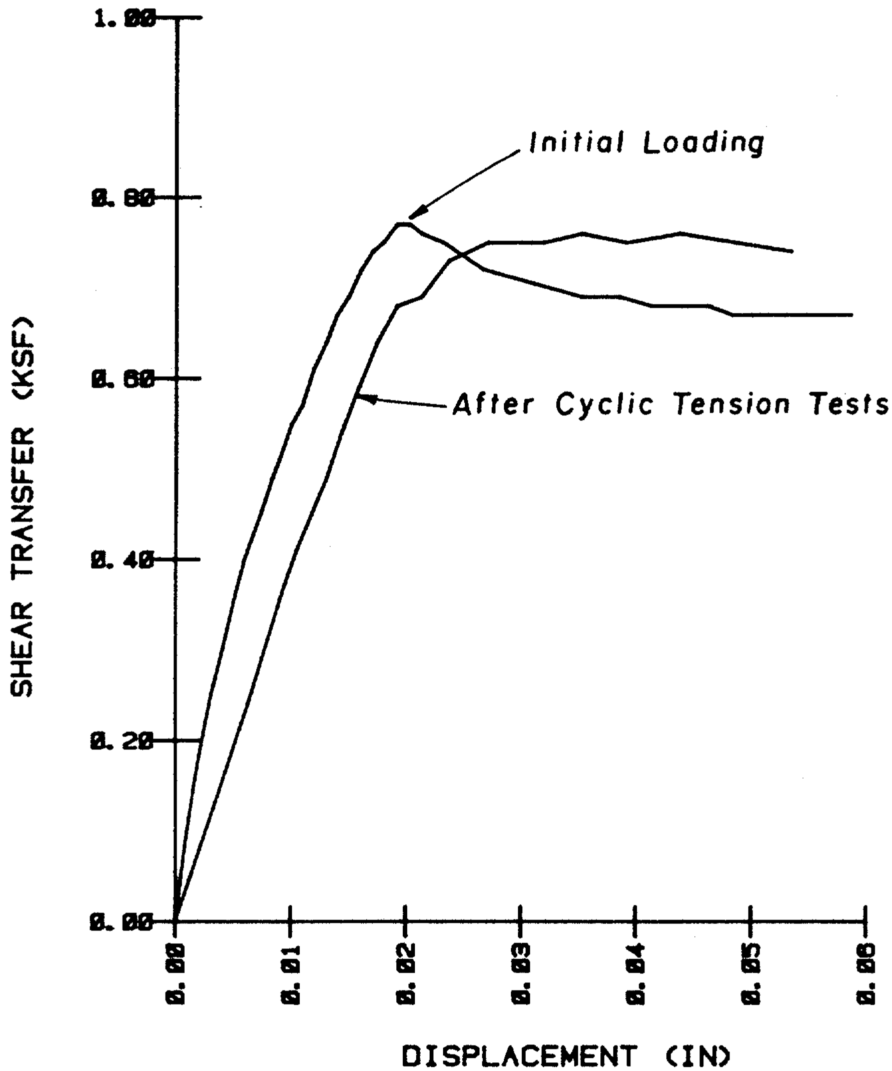
SOIL PRESSURES DURING CONSOLIDATION AT THE 141 - FT DEPTH



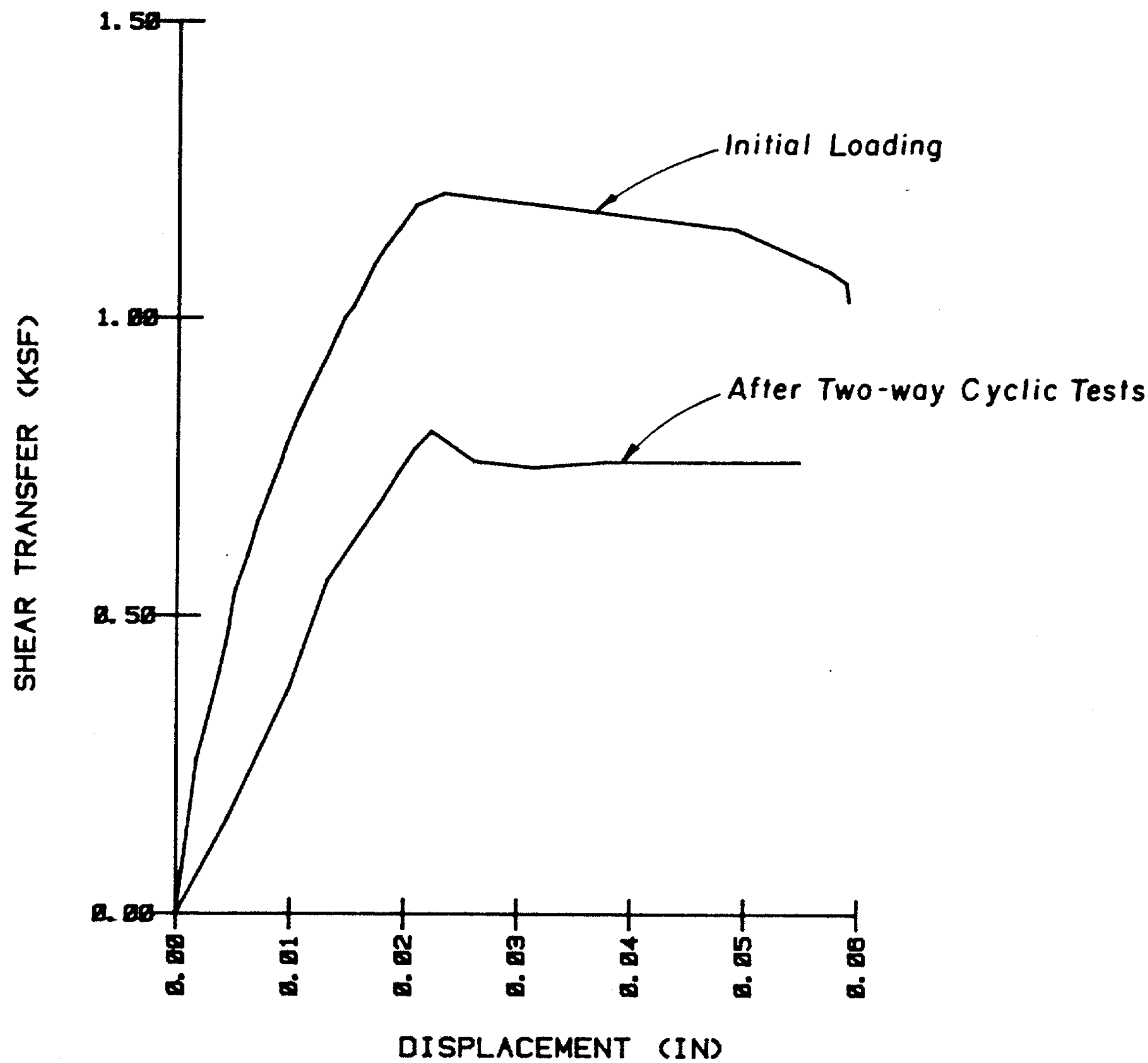
RESULTS OF THE CYCLIC TEST PROGRAM AT THE 141-FT DEPTH
(X-PROBE)



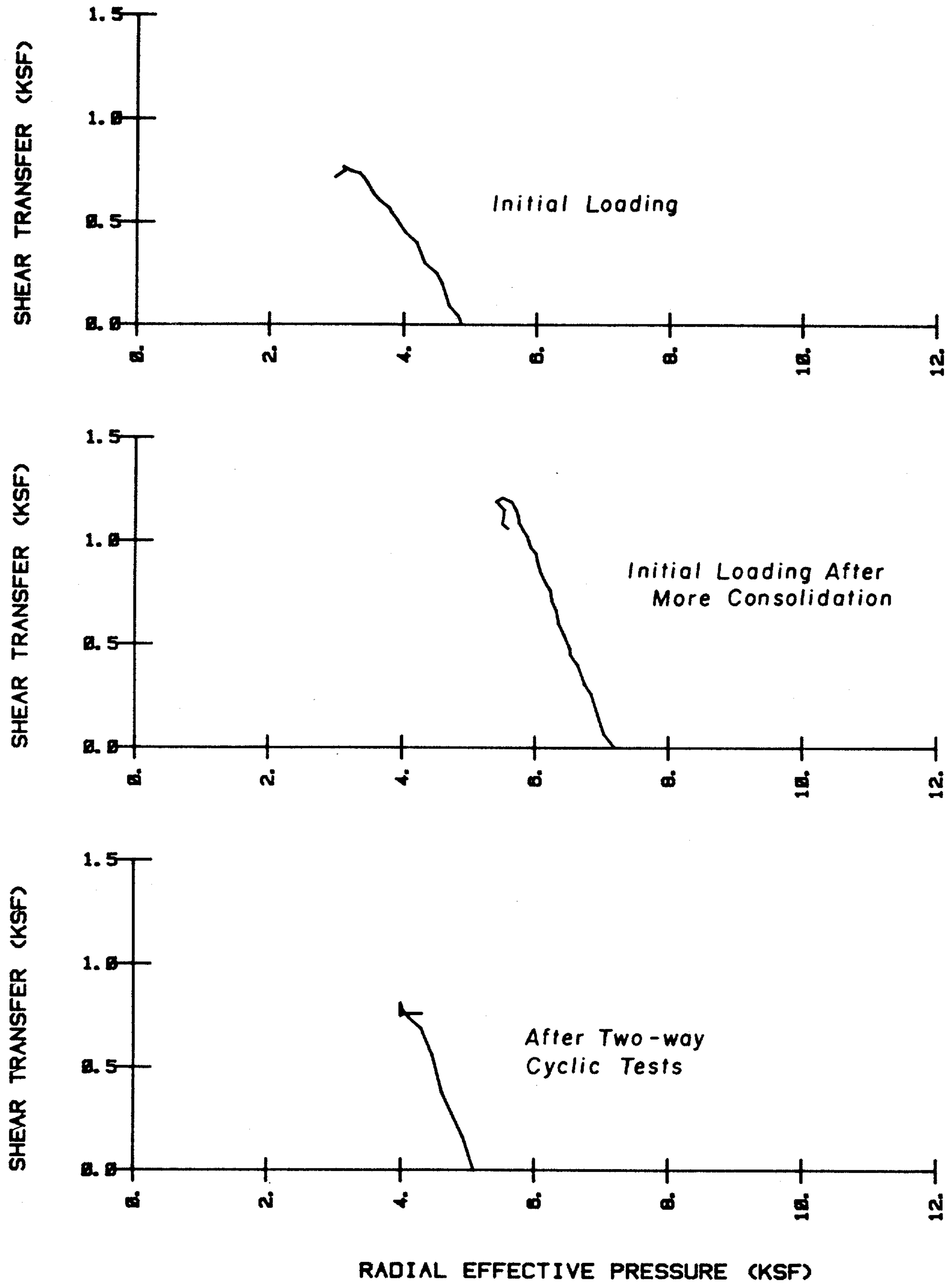
STRESS PATHS DURING LOAD TESTS AT THE 141-FT DEPTH
(X-PROBE)



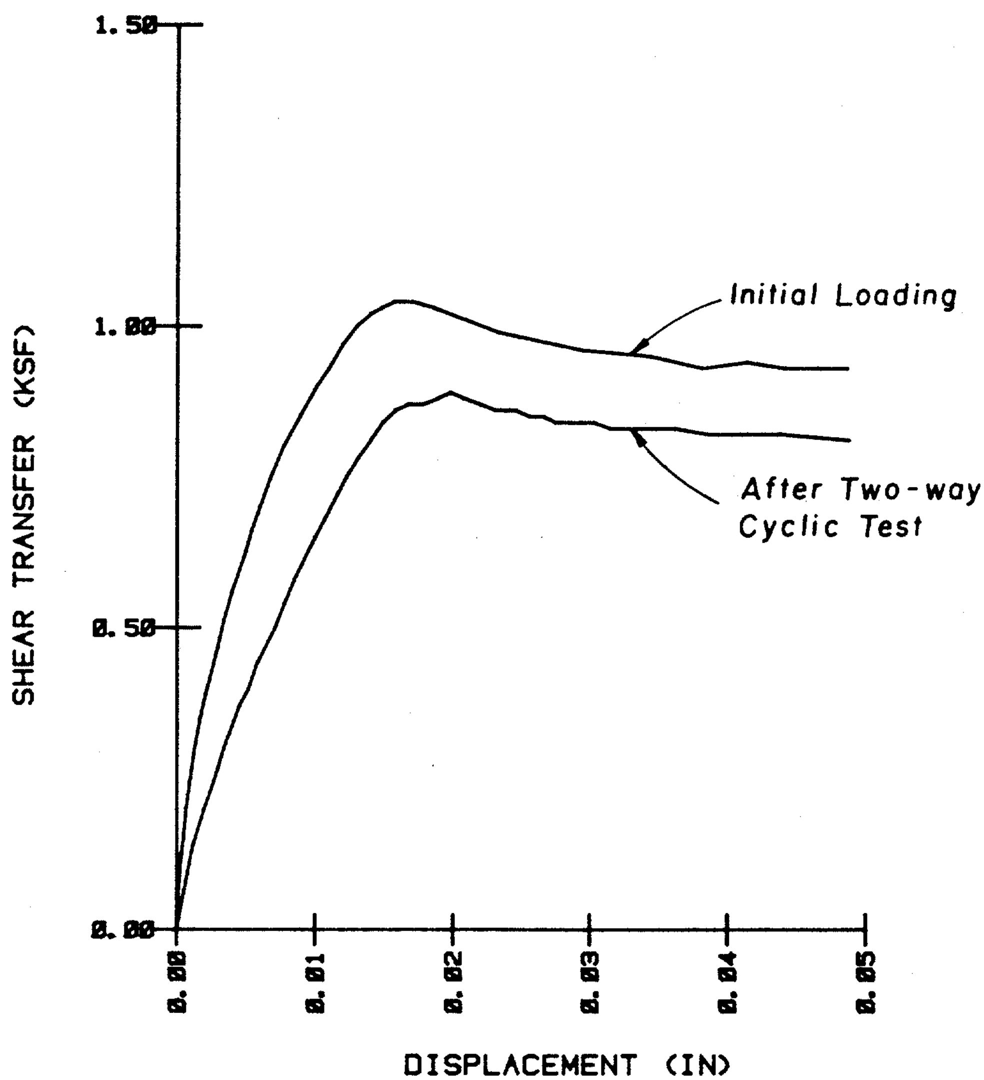
RESULTS OF THE CYCLIC TENSION TESTS
AT THE 141-FT DEPTH (CLOSED-END 3-INCH PROBE)



RESULTS OF THE TWO-WAY CYCLIC TESTS
AT THE 141-FT DEPTH (CLOSED-END 3-INCH PROBE)

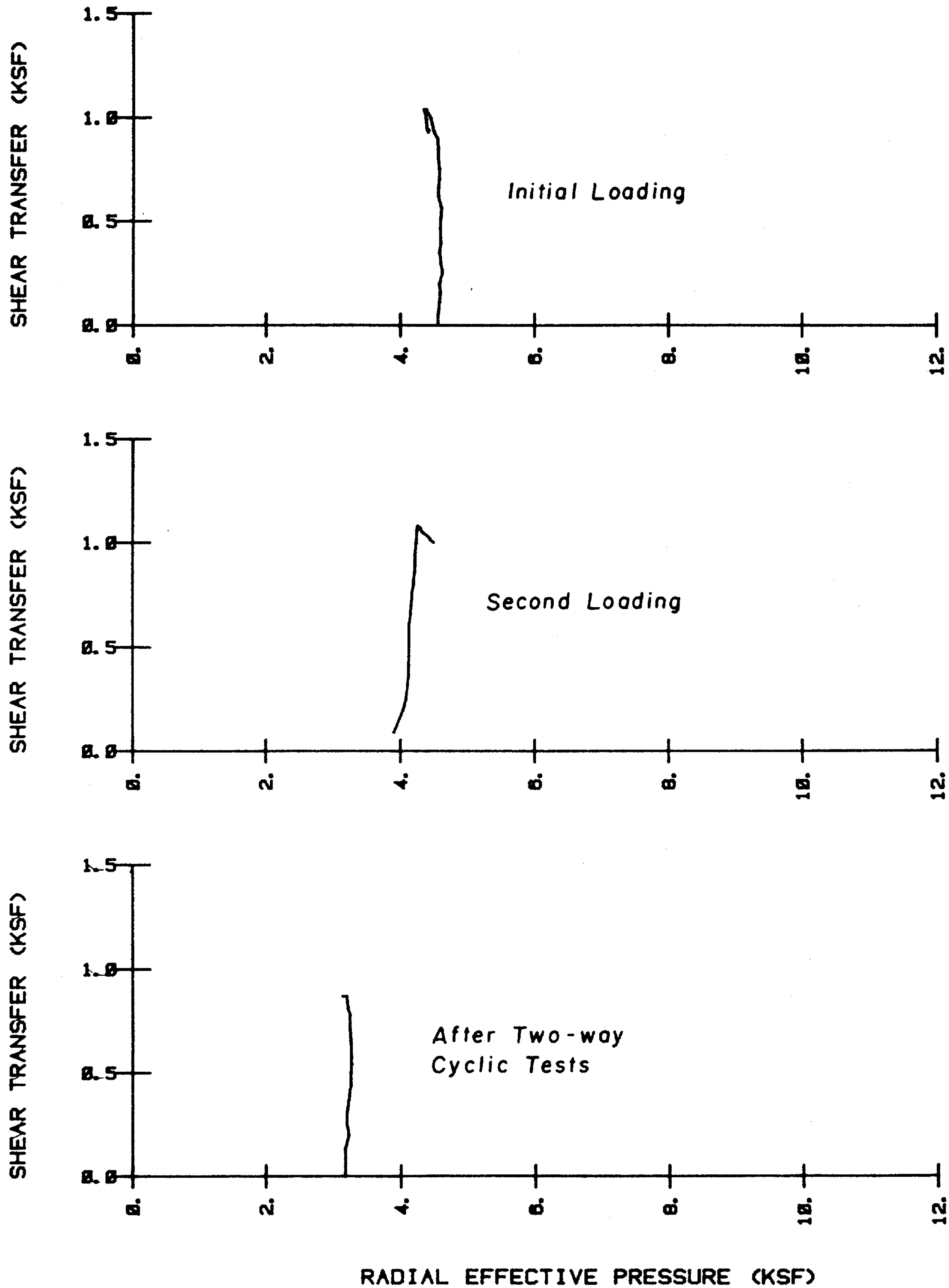


STRESS PATHS DURING LOAD TESTS AT THE 141-FT DEPTH
(CLOSED-END 3-INCH PROBE)



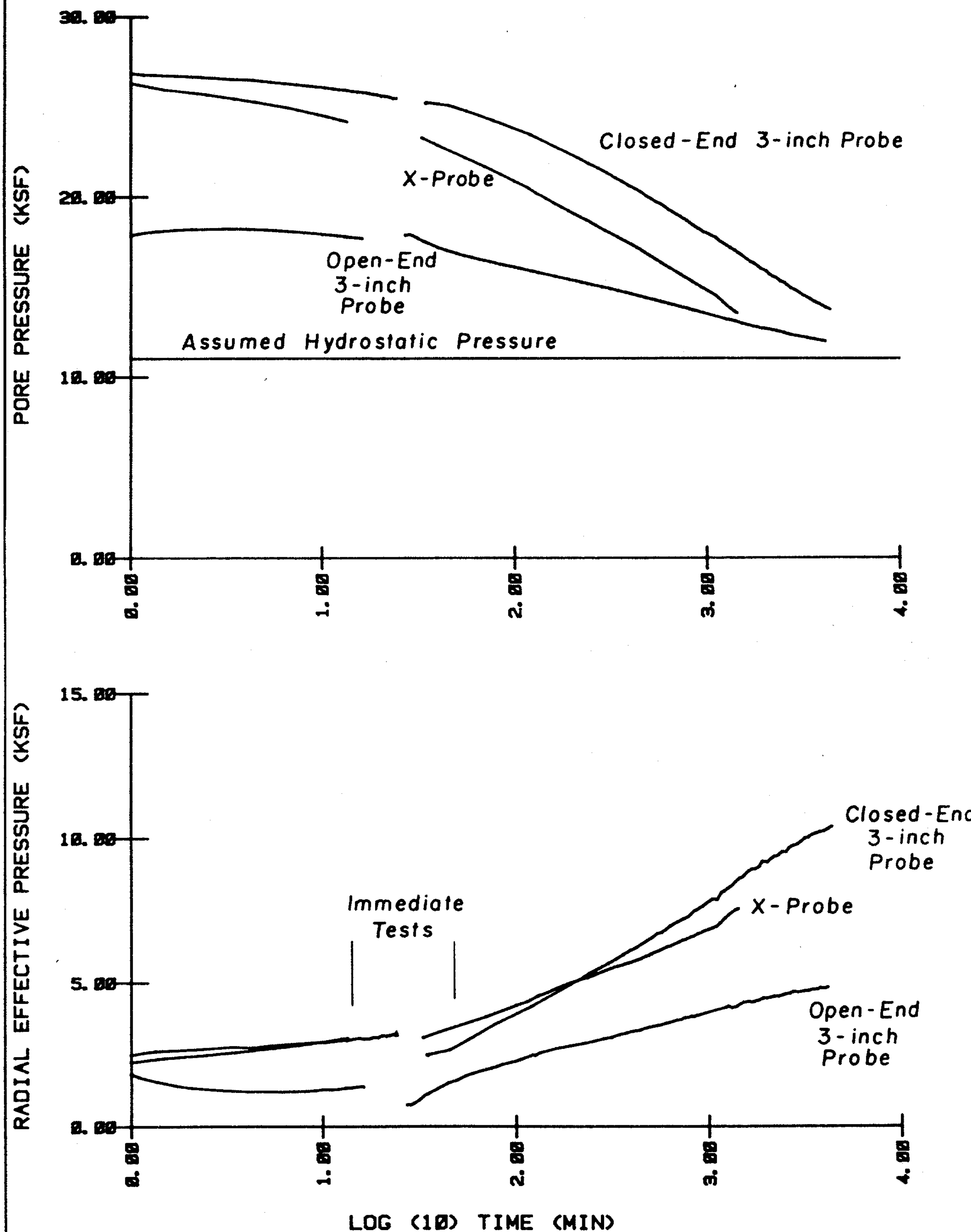
RESULTS OF THE CYCLIC TEST PROGRAM
AT THE 141-FT DEPTH (OPEN-END 3-INCH PROBE)

JOB NO. ______

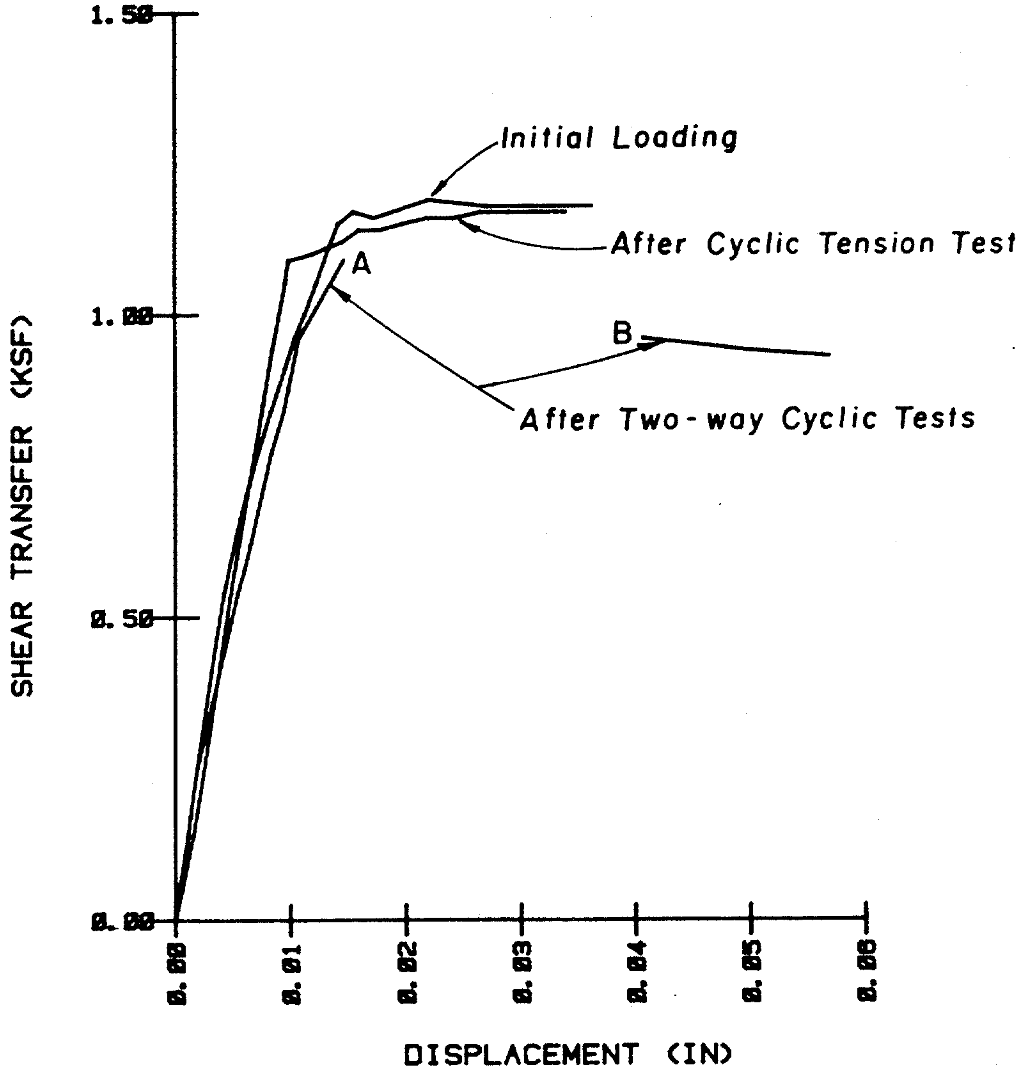


STRESS PATHS DURING LOAD TESTS AT THE 141-FT DEPTH
(OPEN-END 3-INCH PROBE)

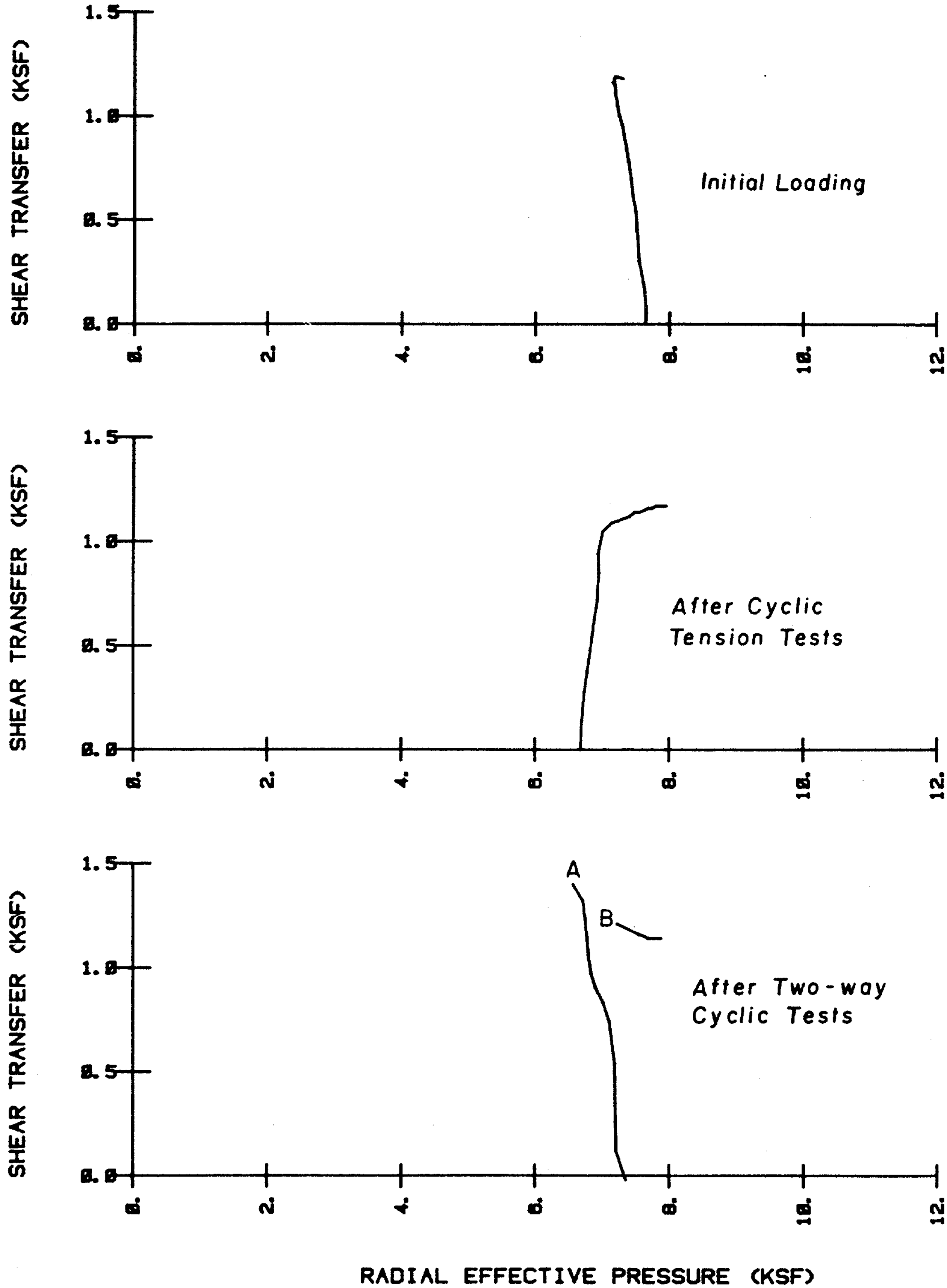
JOB NO.



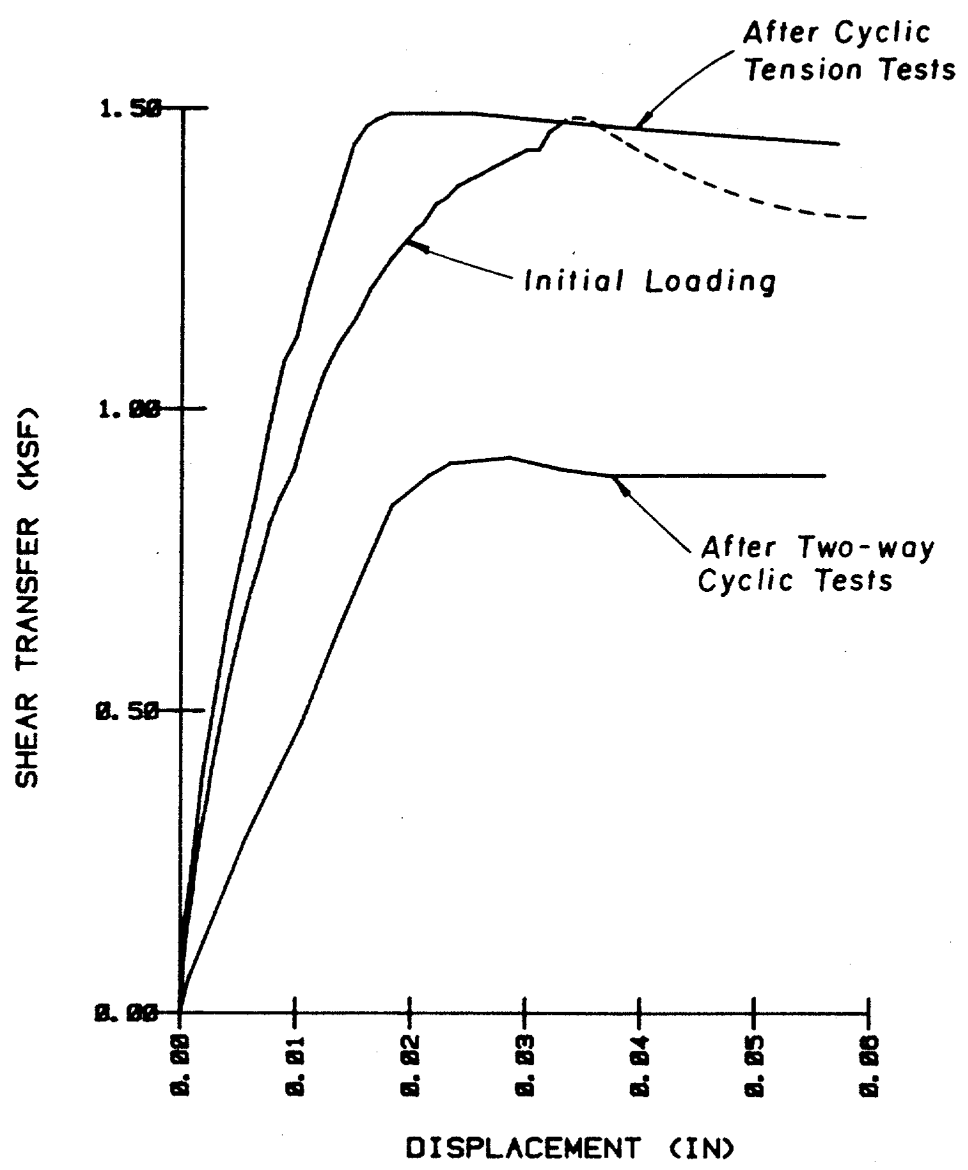
SOIL PRESSURES DURING CONSOLIDATION AT THE 160-FT DEPTH



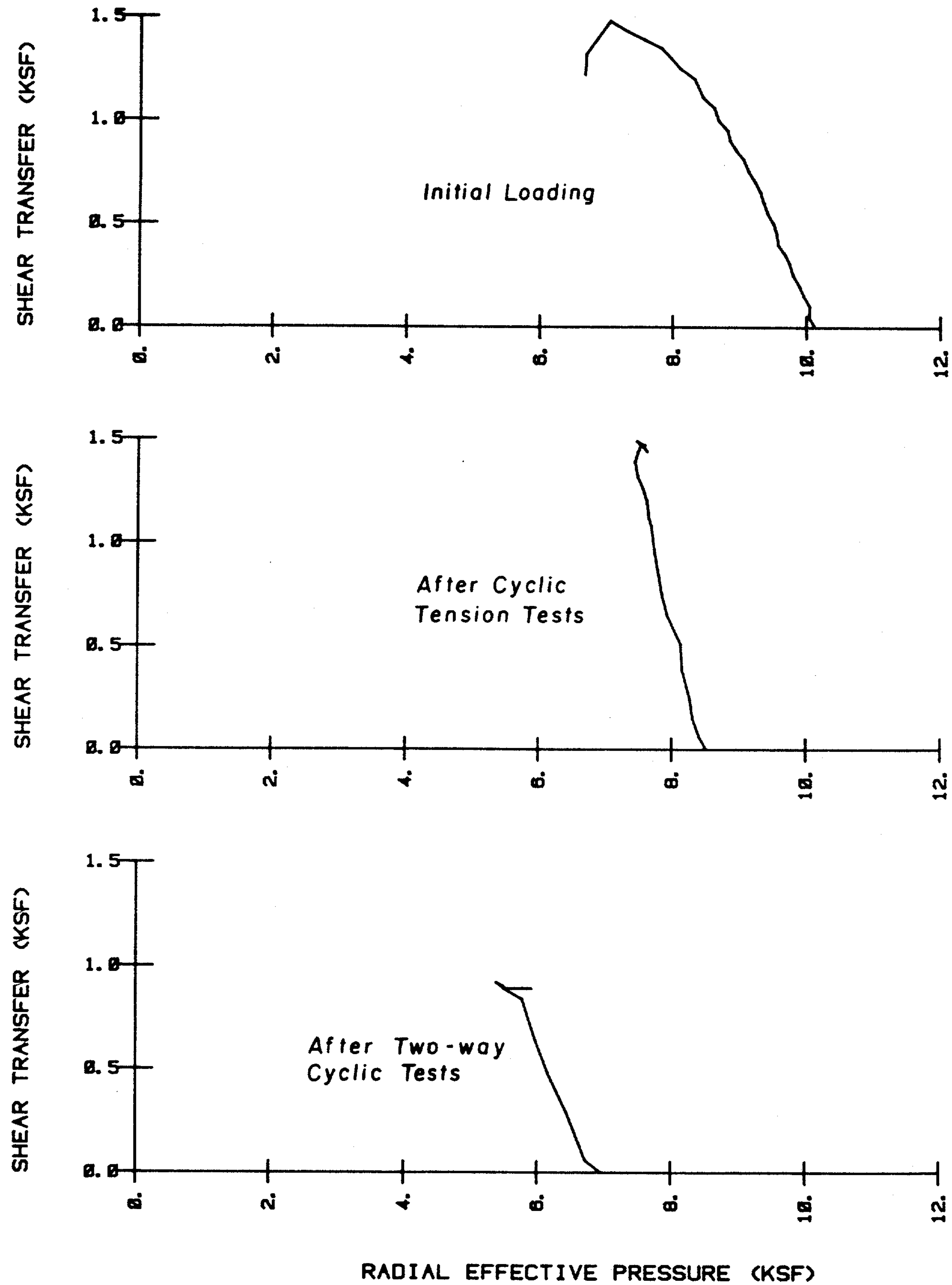
RESULTS OF THE CYCLIC TEST PROGRAM AT THE 160-FT DEPTH
(X-PROBE)



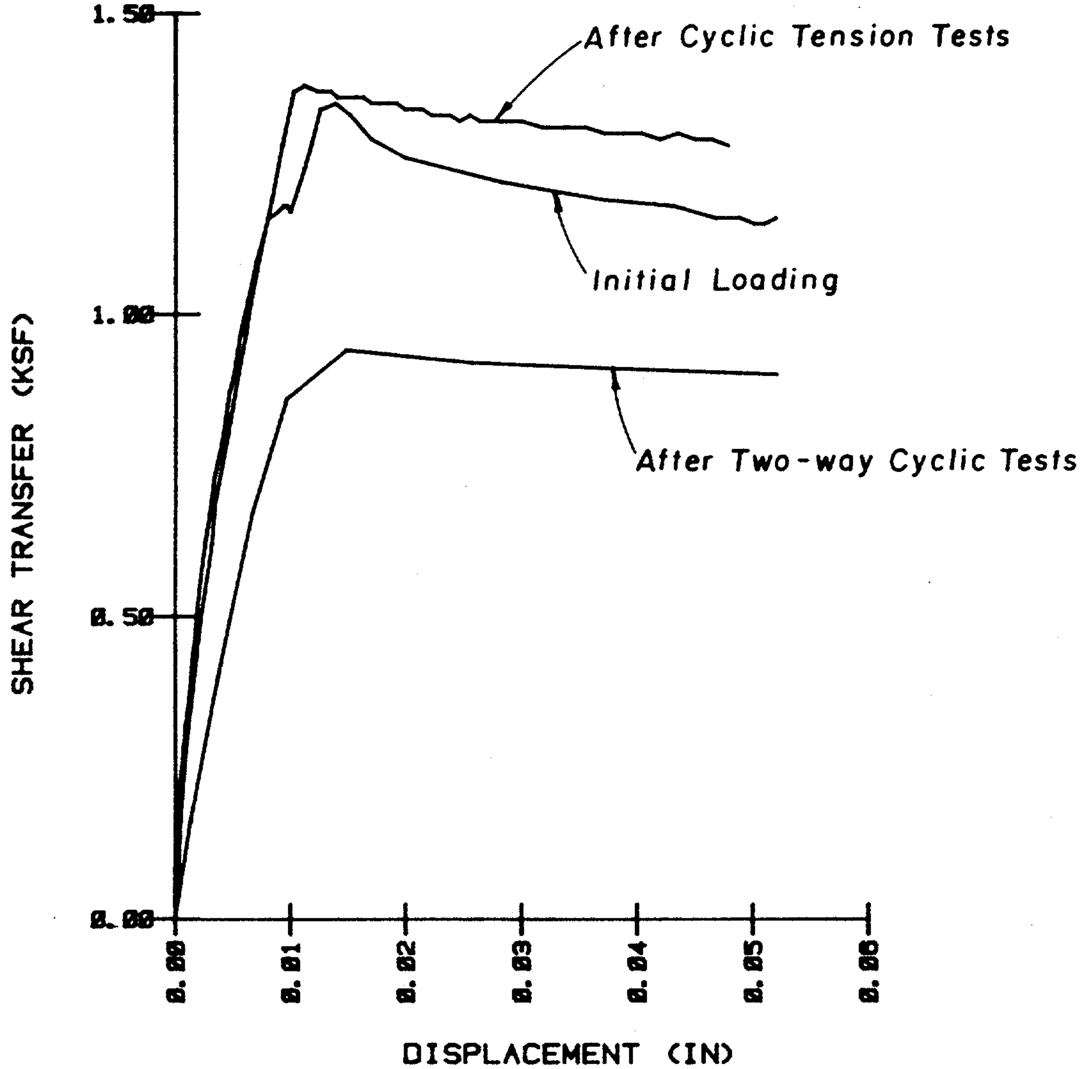
STRESS PATHS DURING LOAD TESTS AT THE 160-FT DEPTH
(X-PROBE)



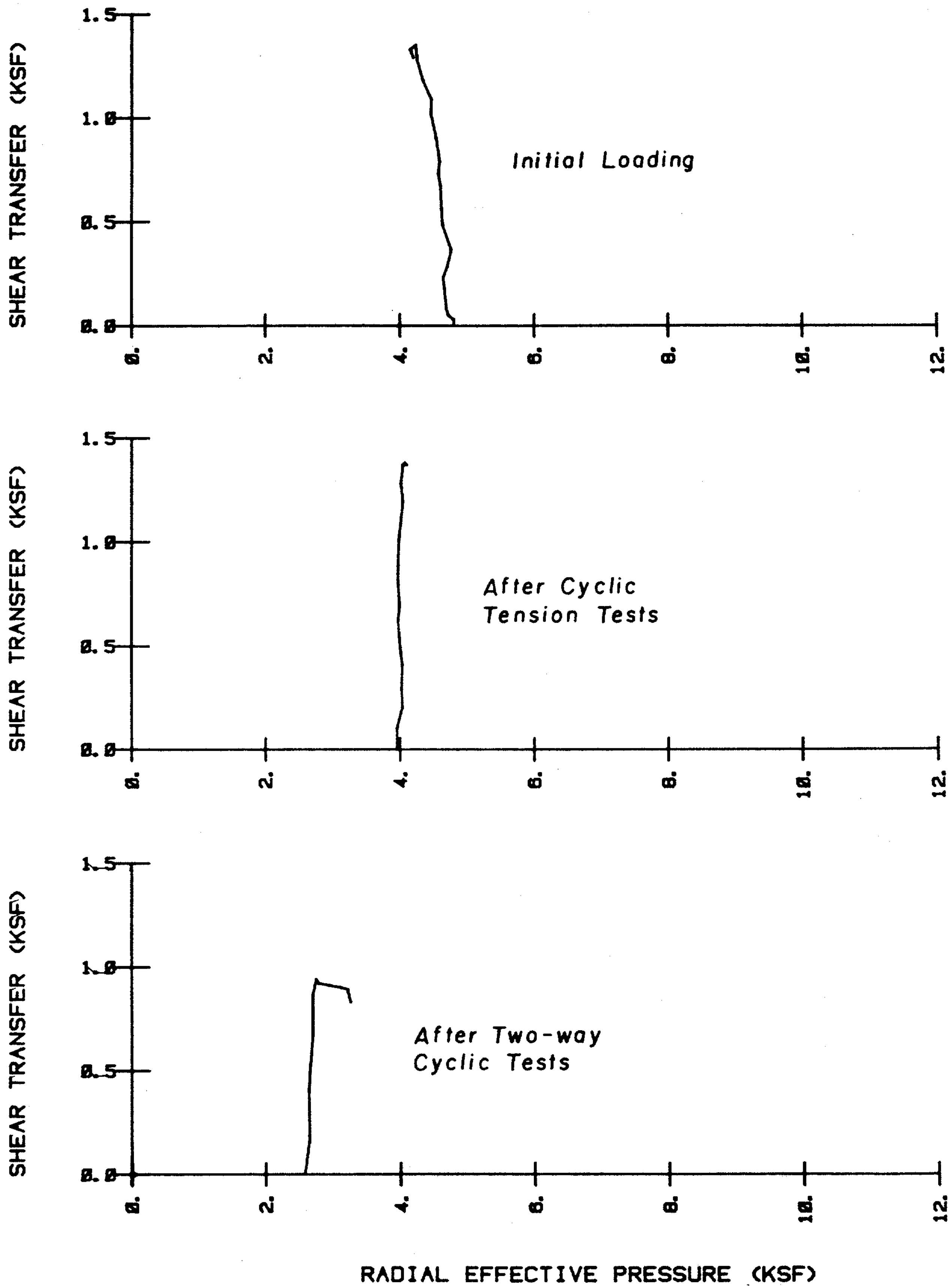
RESULTS OF THE CYCLIC TEST PROGRAM
AT THE 160-FT DEPTH (CLOSED-END 3-INCH PROBE)



STRESS PATHS DURING LOAD TESTS AT THE 160-FT DEPTH
(CLOSED-END 3-INCH PROBE)



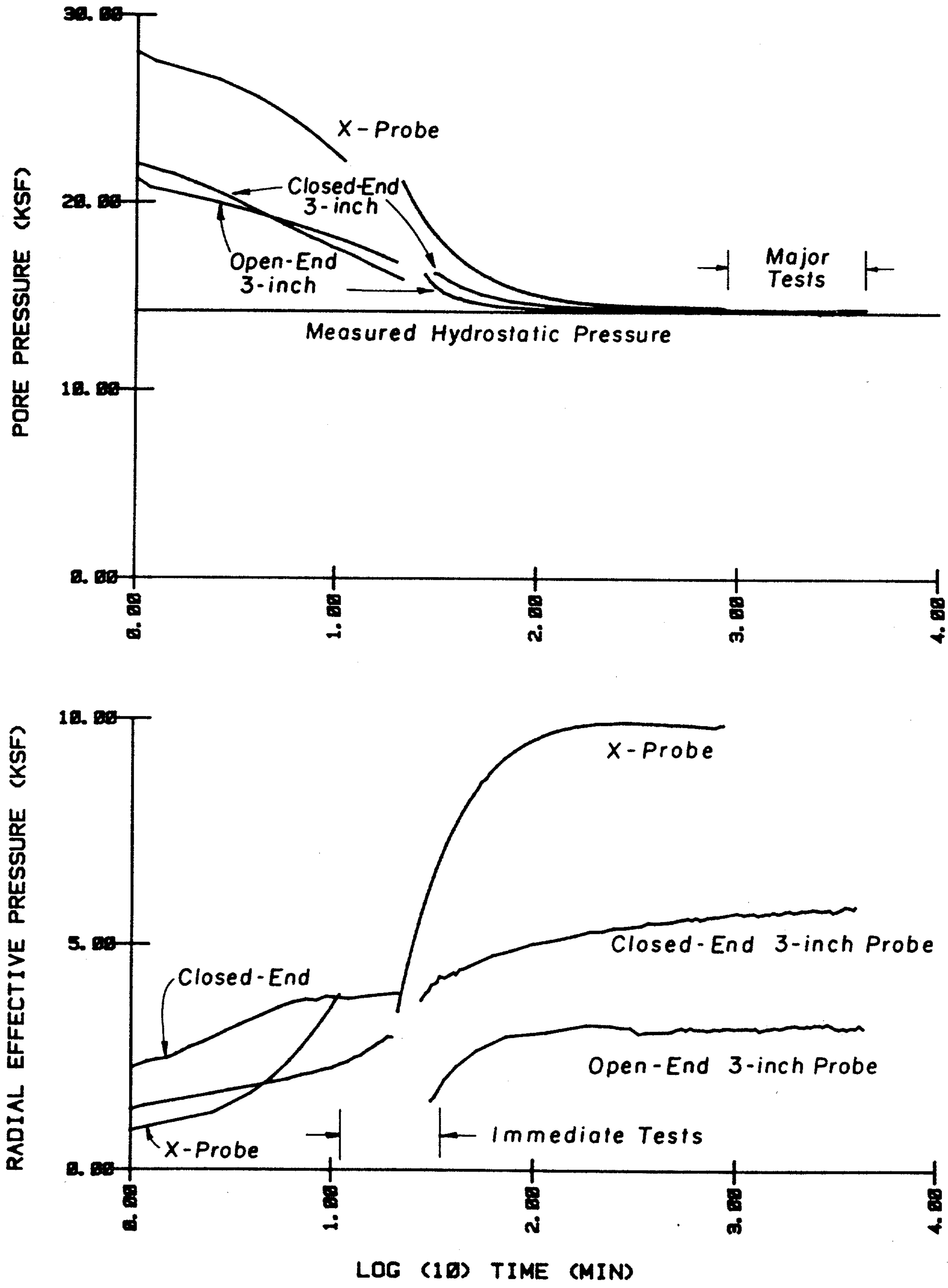
RESULTS OF THE CYCLIC TEST PROGRAM
AT THE 160-FT DEPTH (OPEN-END 3-INCH PROBE)



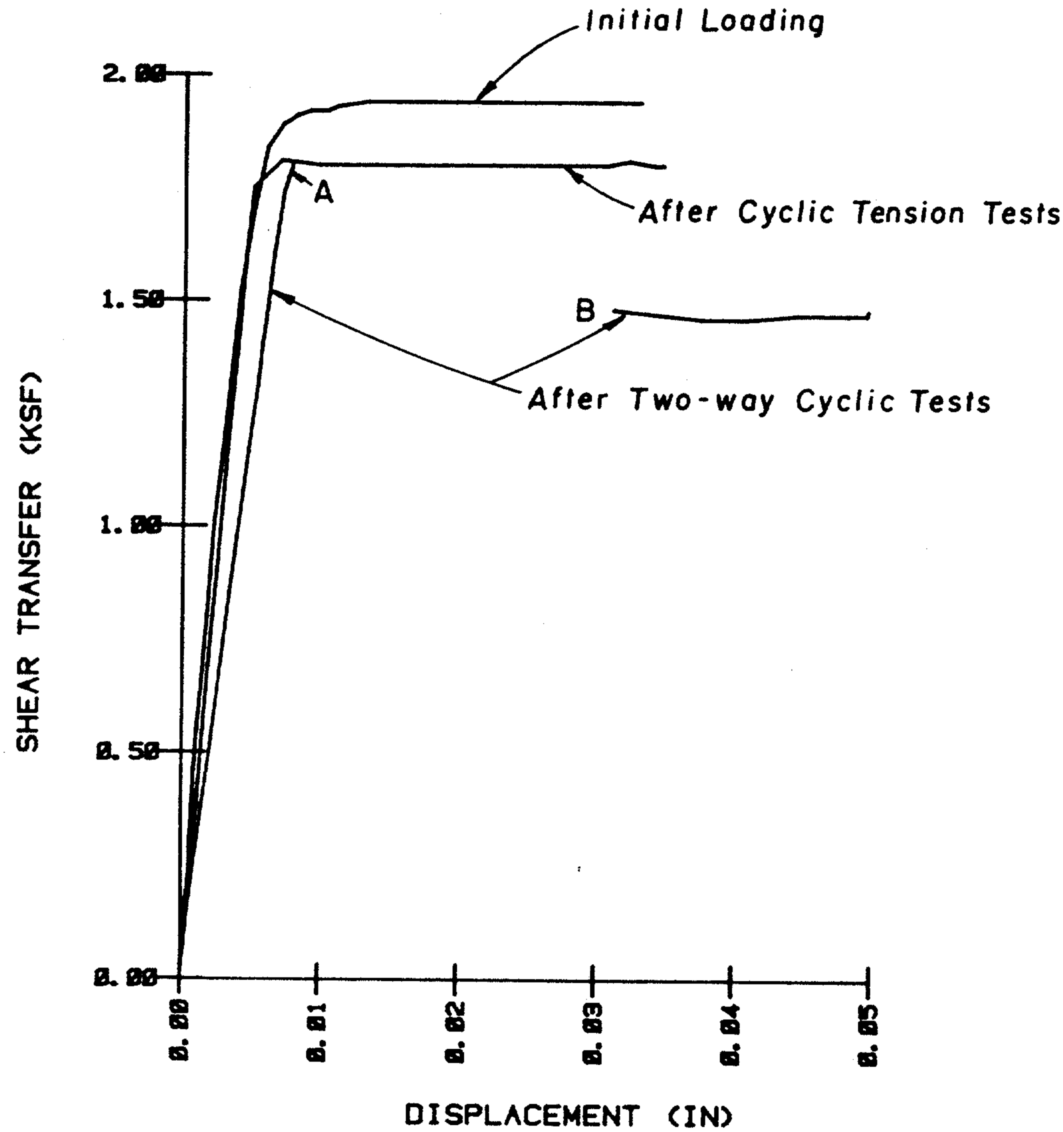
STRESS PATHS DURING LOAD TESTS AT THE 160-FT DEPTH
(OPEN-END 3-INCH PROBE)

The Earth Technology Corporation

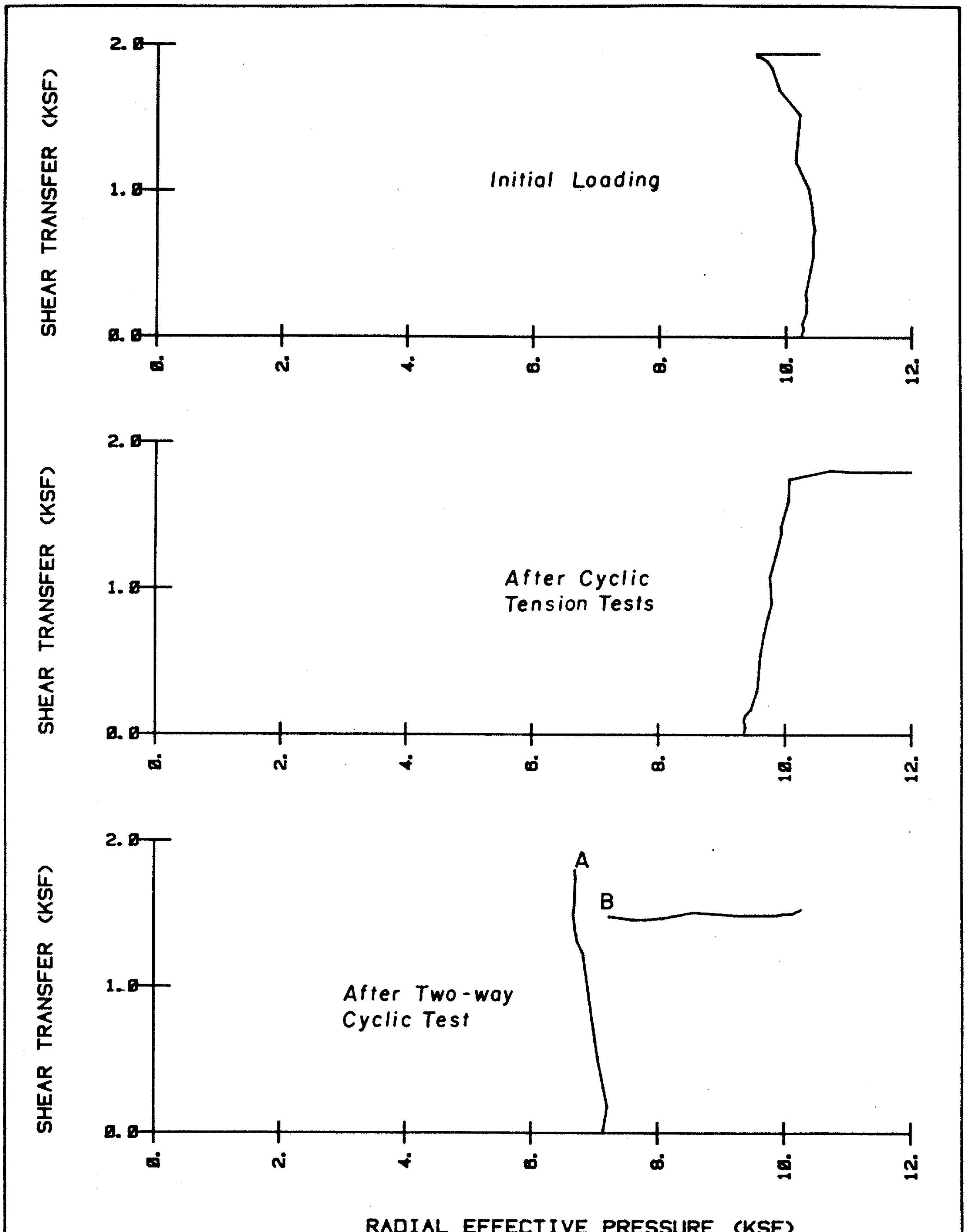
JOB NO.



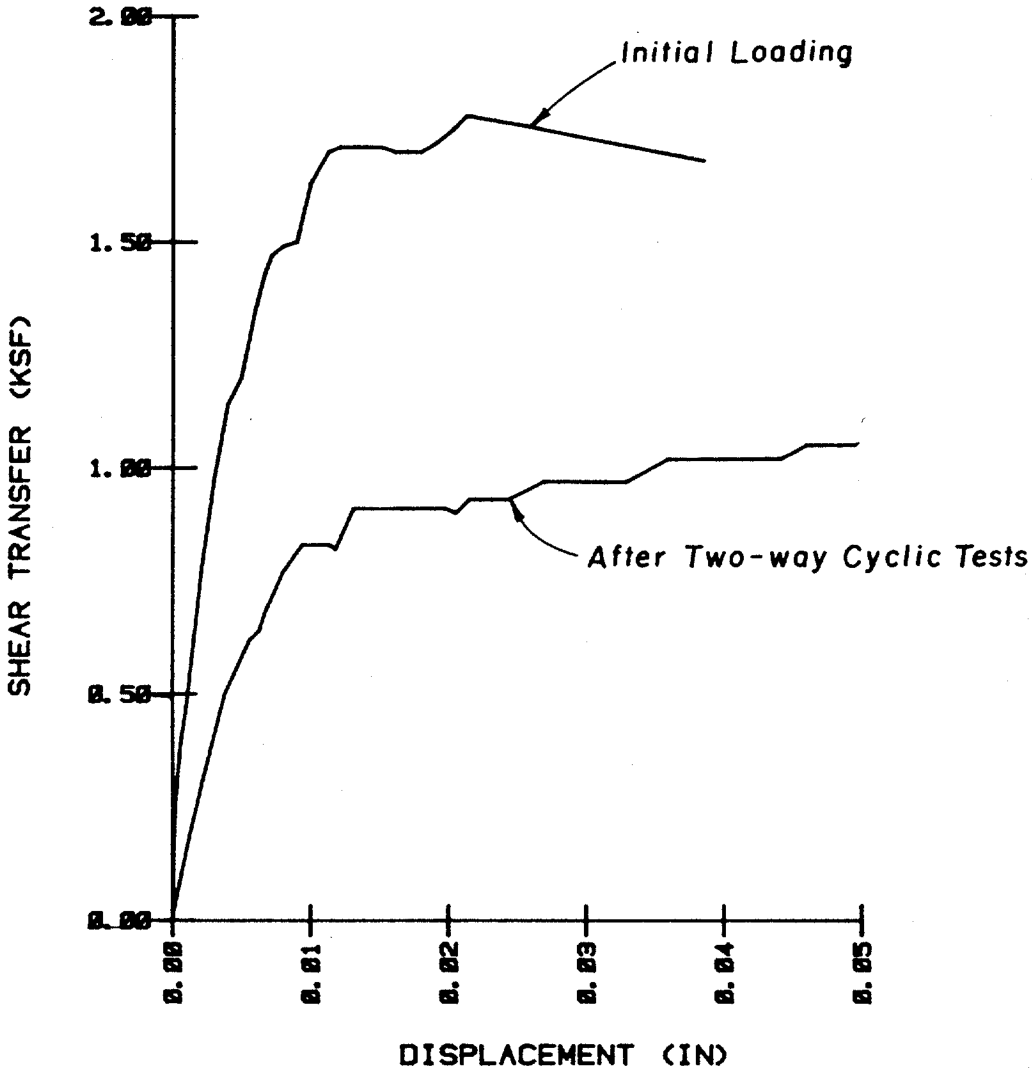
SOIL PRESSURES DURING CONSOLIDATION AT THE 210-FT DEPTH



RESULTS OF THE CYCLIC TEST PROGRAM AT THE 210-FT DEPTH
(X-PROBE)

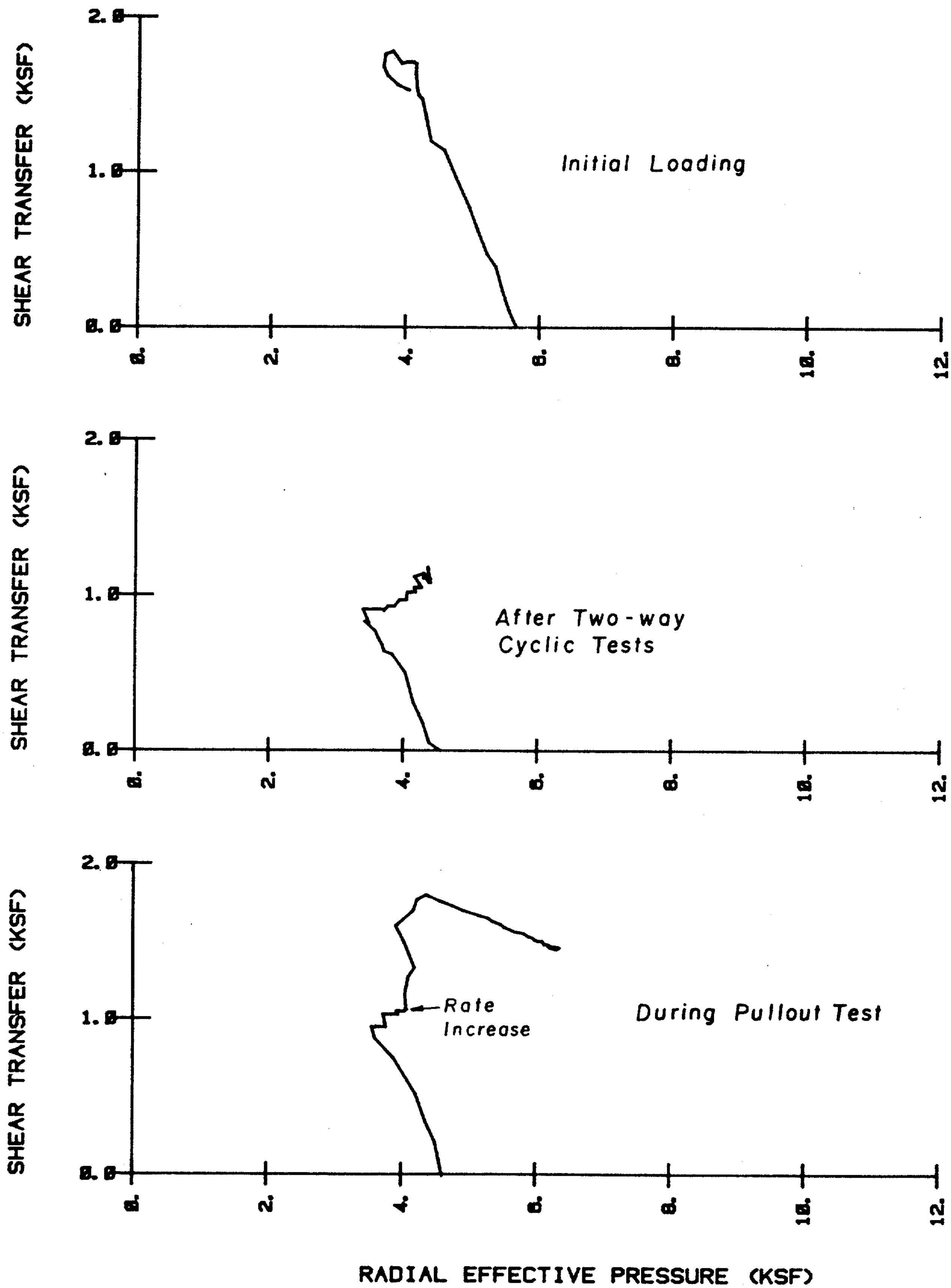


STRESS PATHS DURING LOAD TESTS AT THE 210-FT DEPTH
(X-PROBE)

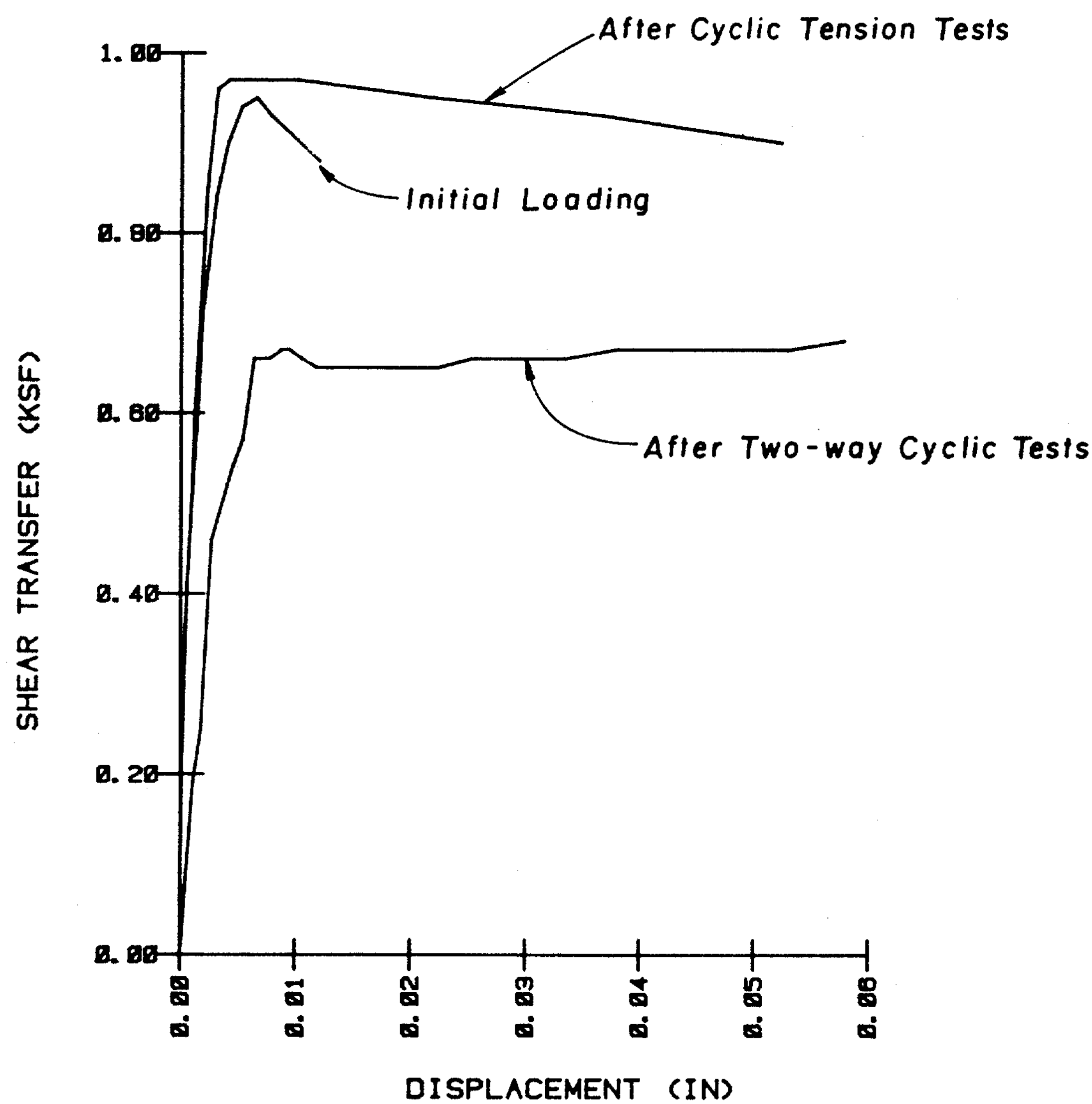


RESULTS OF THE CYCLIC TEST PROGRAM
AT THE 210-FT DEPTH (CLOSED-END 3-INCH PROBE)

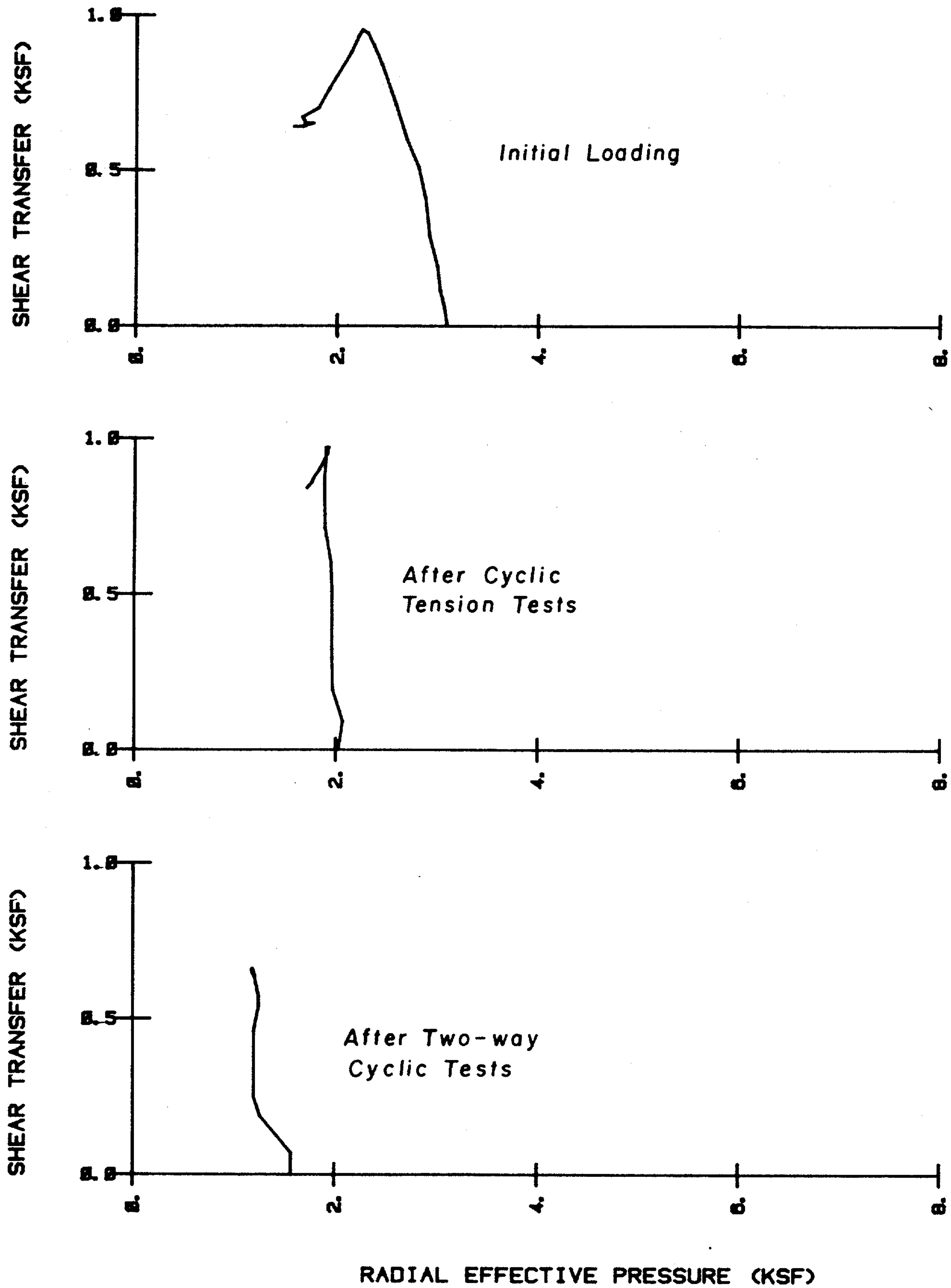
JOB NO.



STRESS PATHS DURING LOAD TESTS AT THE 210-FT DEPTH
(CLOSED-END 3-INCH PROBE)

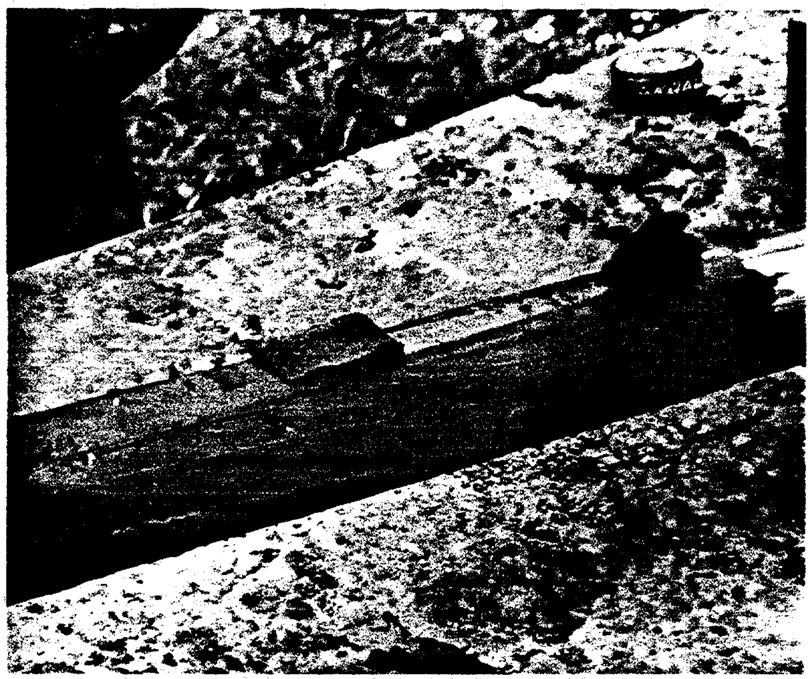


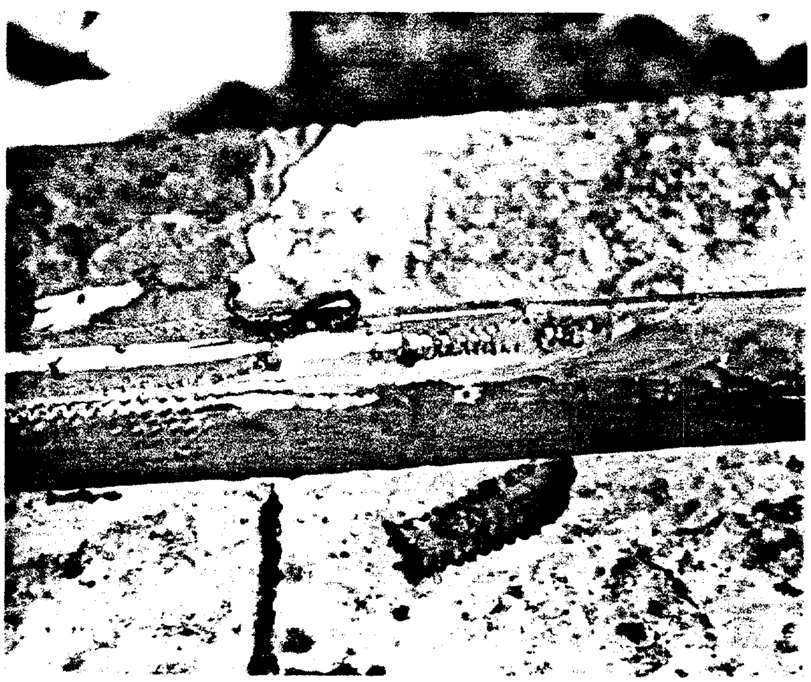
RESULTS OF THE CYCLIC TEST PROGRAM
AT THE 210-FT DEPTH (OPEN-END 3-INCH PROBE)



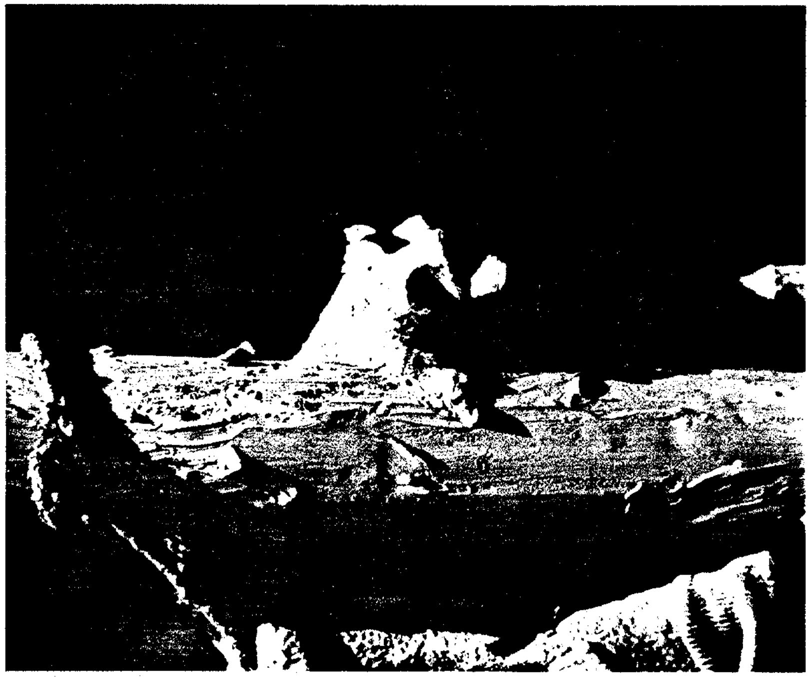
STRESS PATHS DURING LOAD TESTS AT THE 210-FT DEPTH
(OPEN-END 3-INCH PROBE)

The Earth Technology Corporation

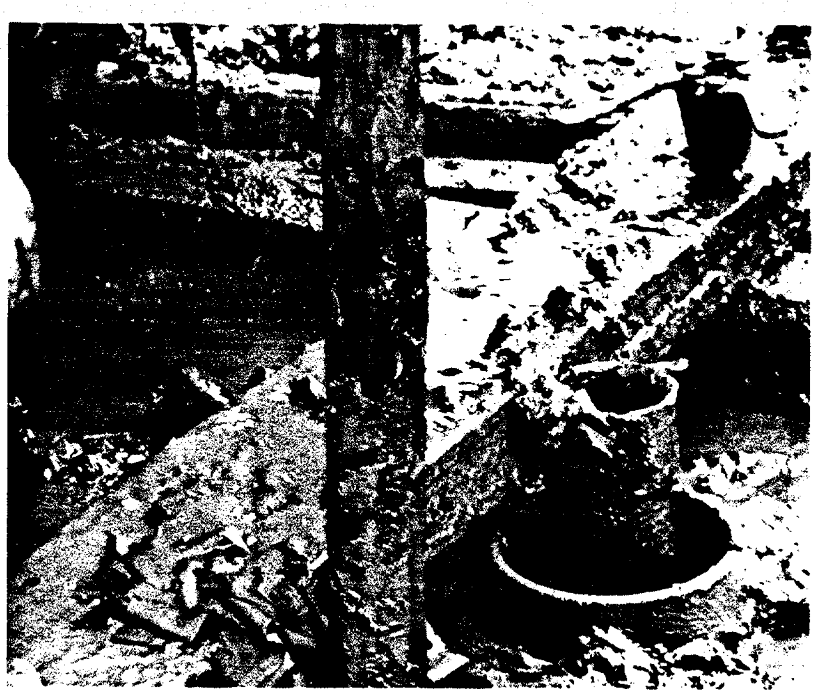




Instrumented Section of 3-Inch Probes After Removal from Experiments in Clay Soils

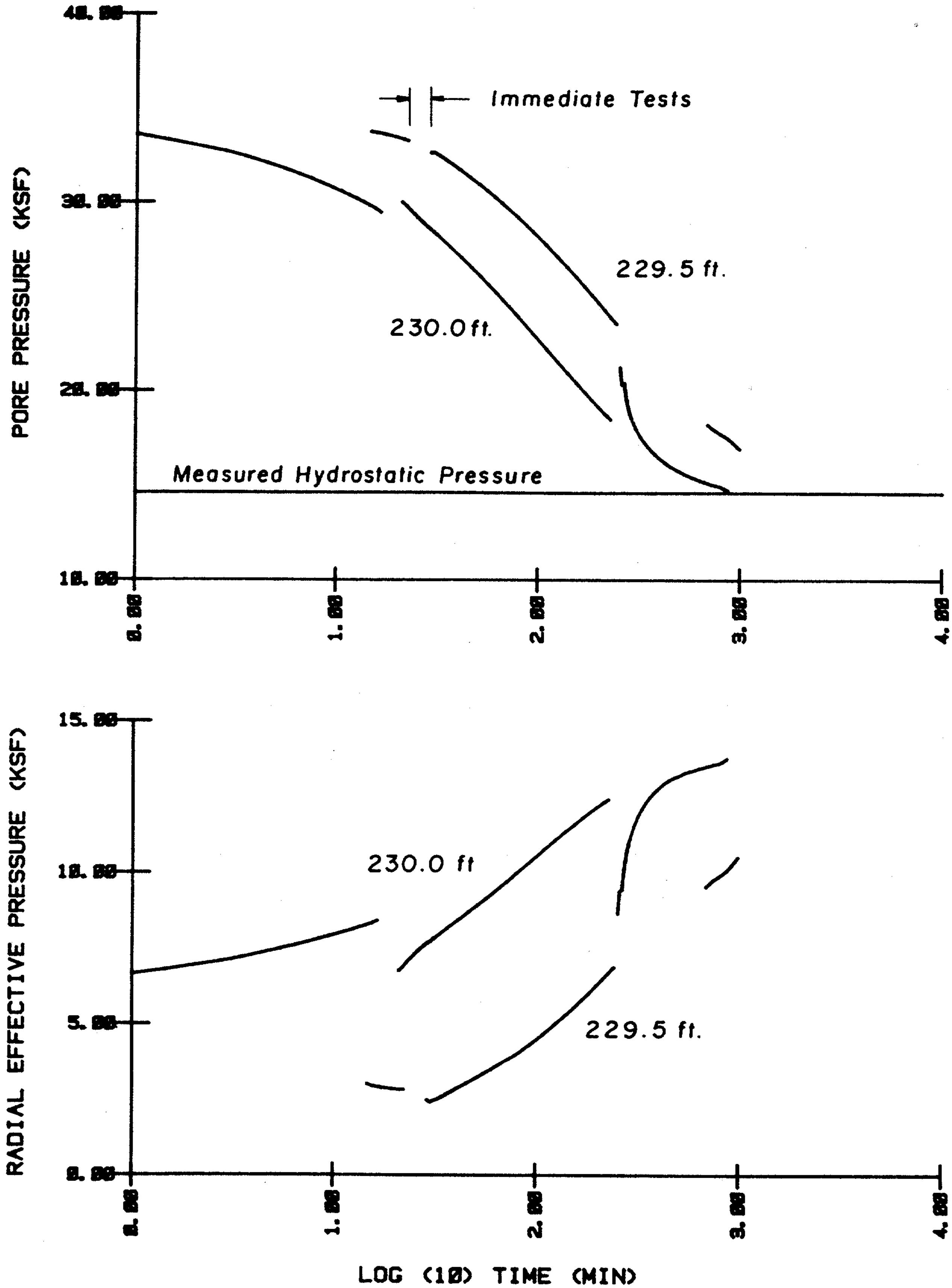


a. Soft Clay



b. Firm-to-Stiff Clay

Cutting Shoe after Experiments in Clay Soils



SOIL PRESSURES DURING CONSOLIDATION

AT THE 229.5 AND 230-FT DEPTH

The Earth Technology Corporation

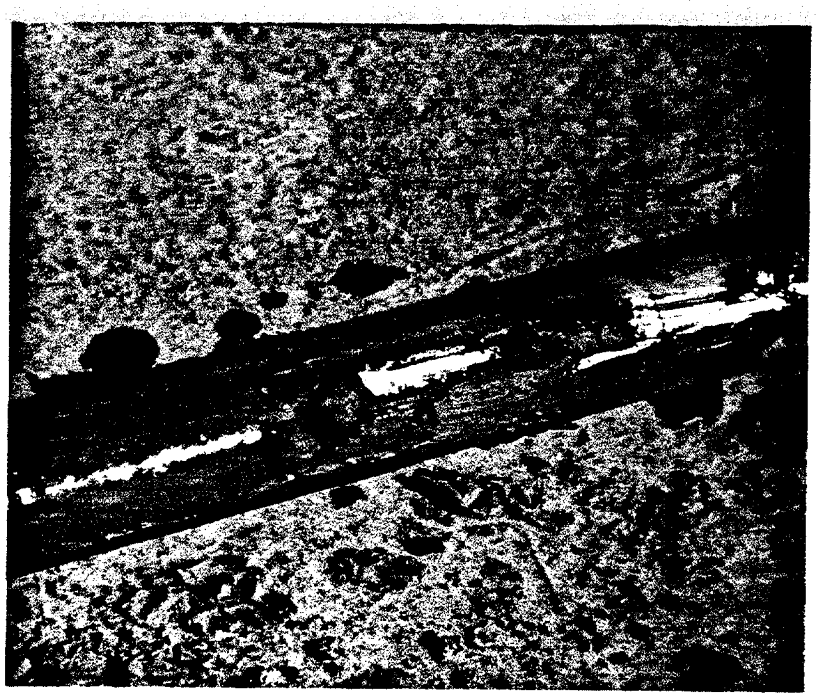


Plate 42. X-Probe after Removal from 229.5-Ft Depth

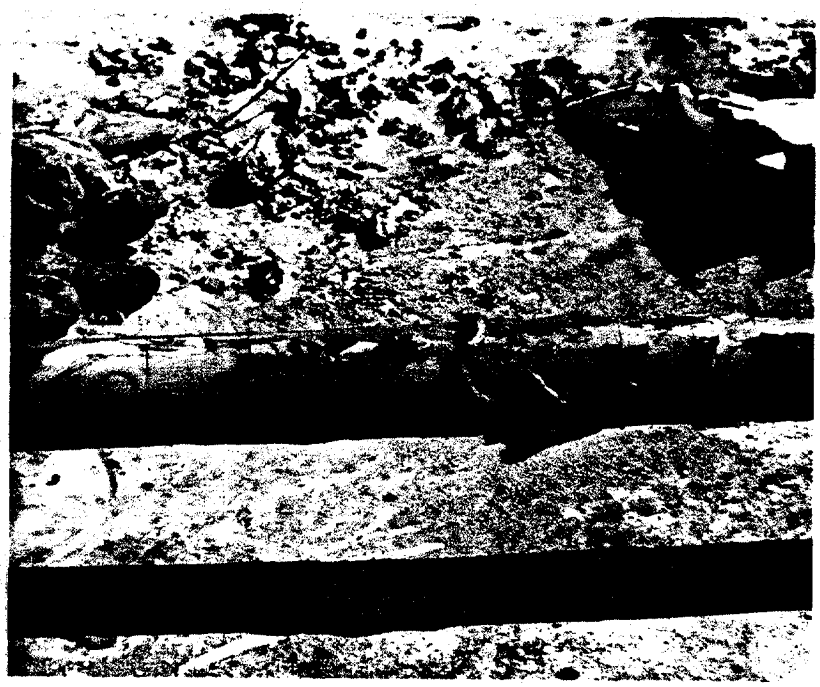
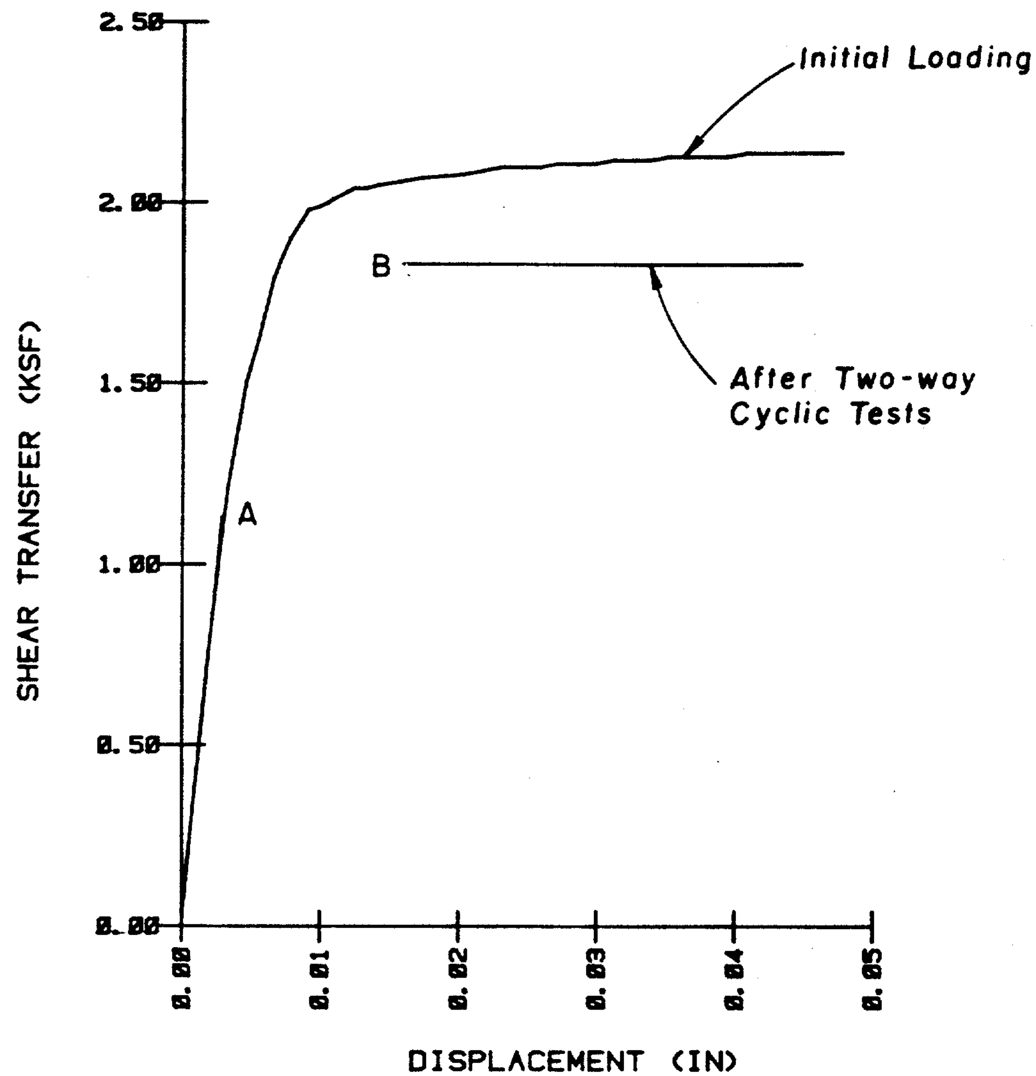
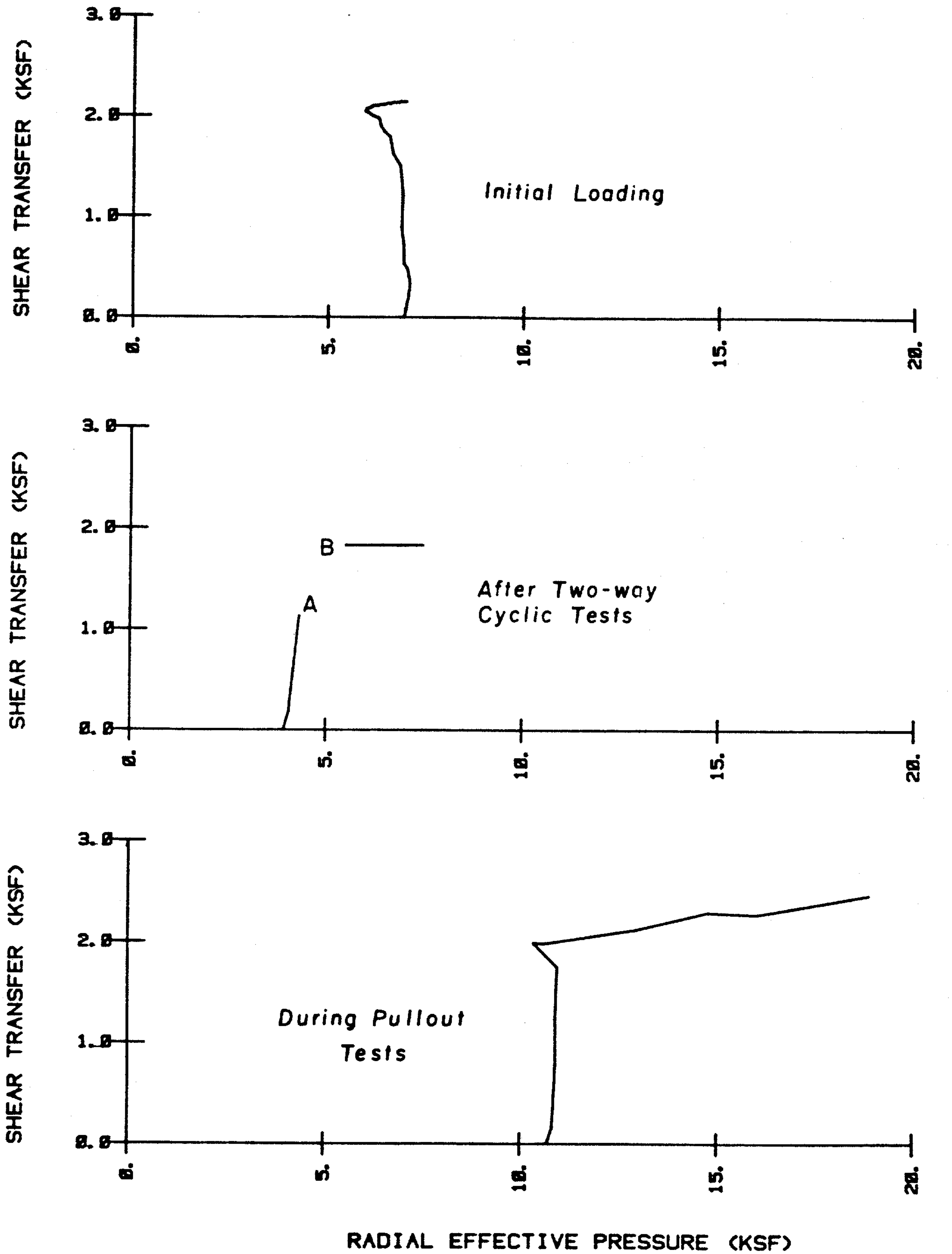


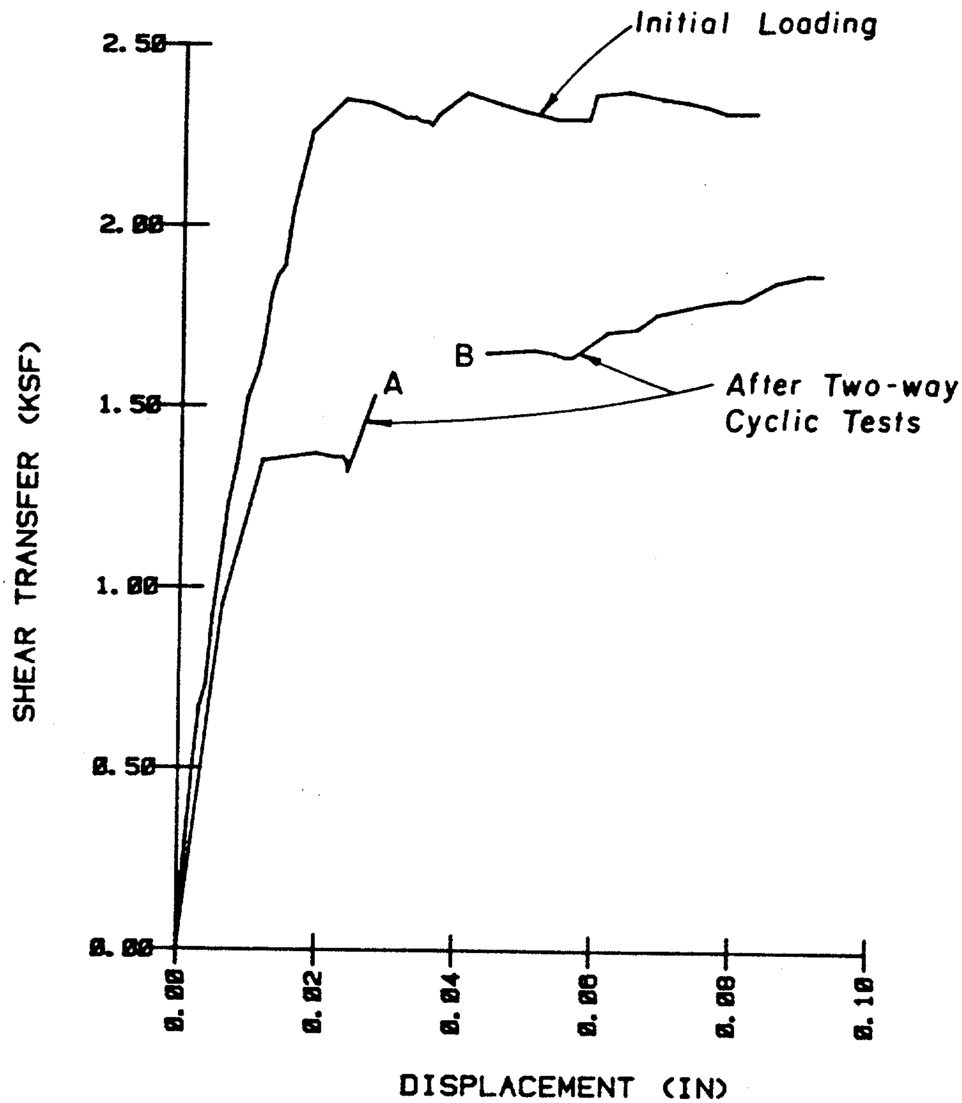
Plate 43. X-Probe after Removal from 230-Ft Depth



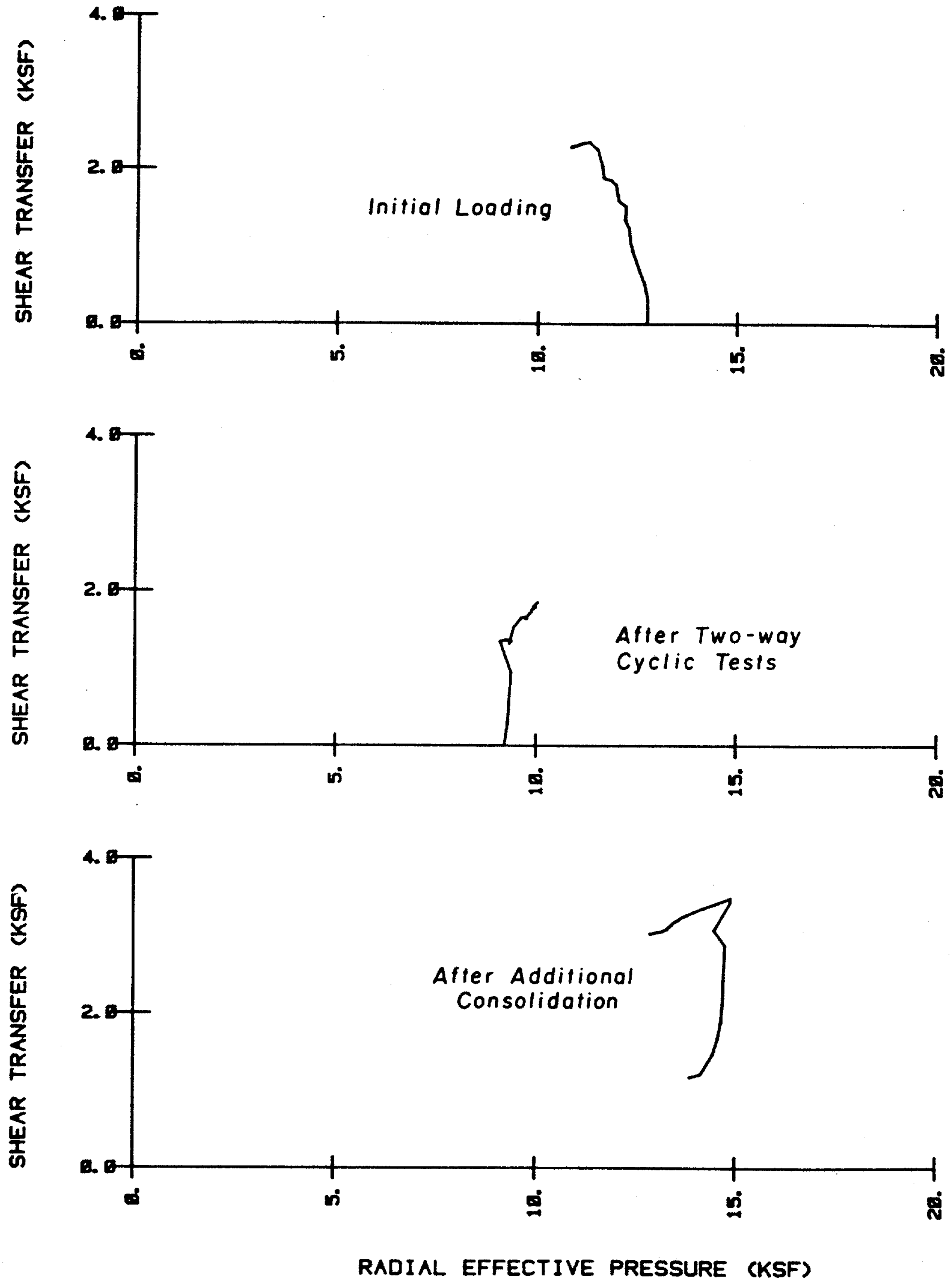
RESULTS OF THE CYCLIC TEST PROGRAM AT THE 229.5-FT DEPTH
(X-PROBE)



STRESS PATHS DURING LOAD TESTS AT THE 229.5-FT DEPTH
(X-PROBE)

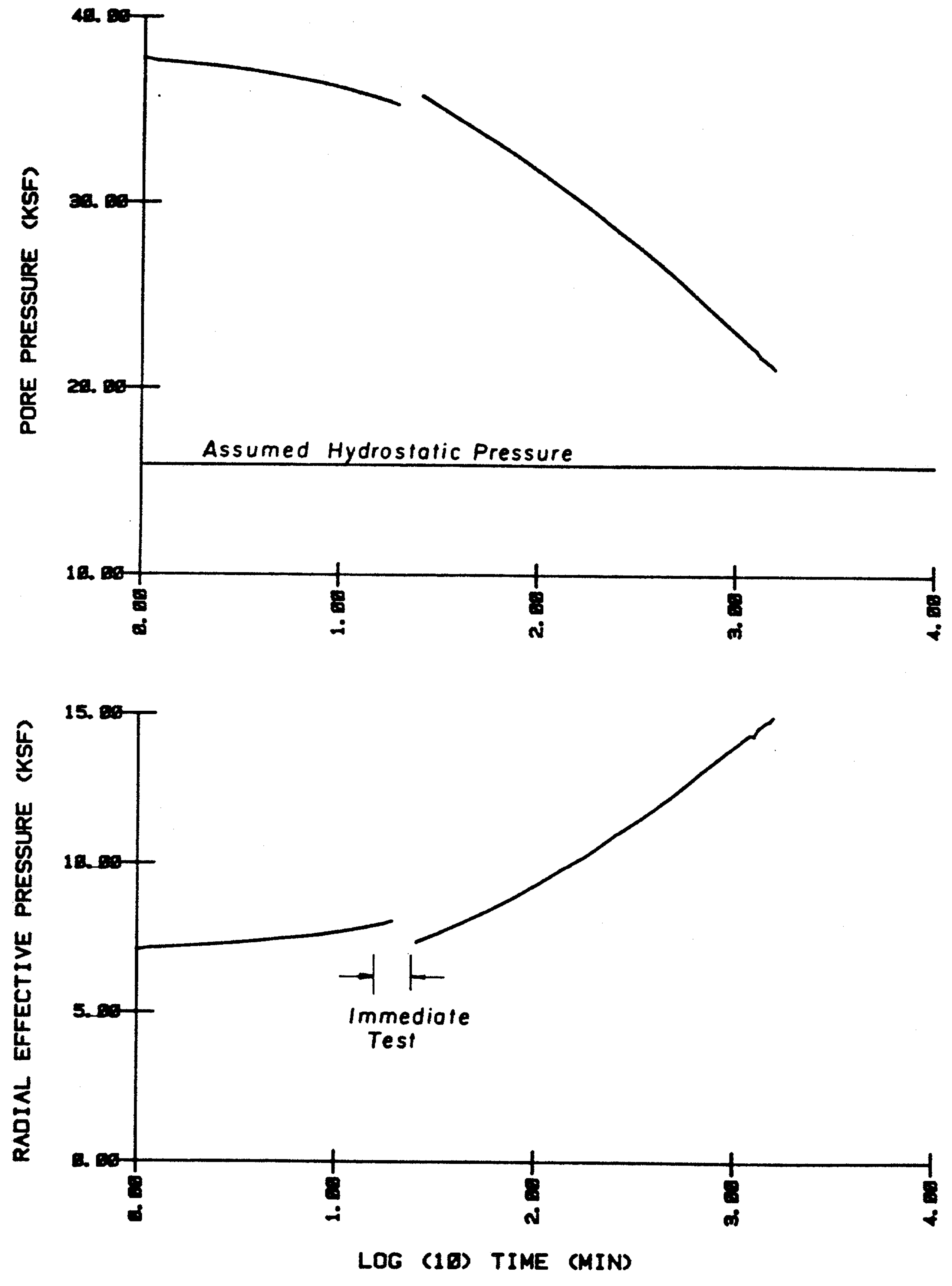


RESULTS OF THE CYCLIC TEST PROGRAM AT THE 230-FT DEPTH
(X-PROBE)



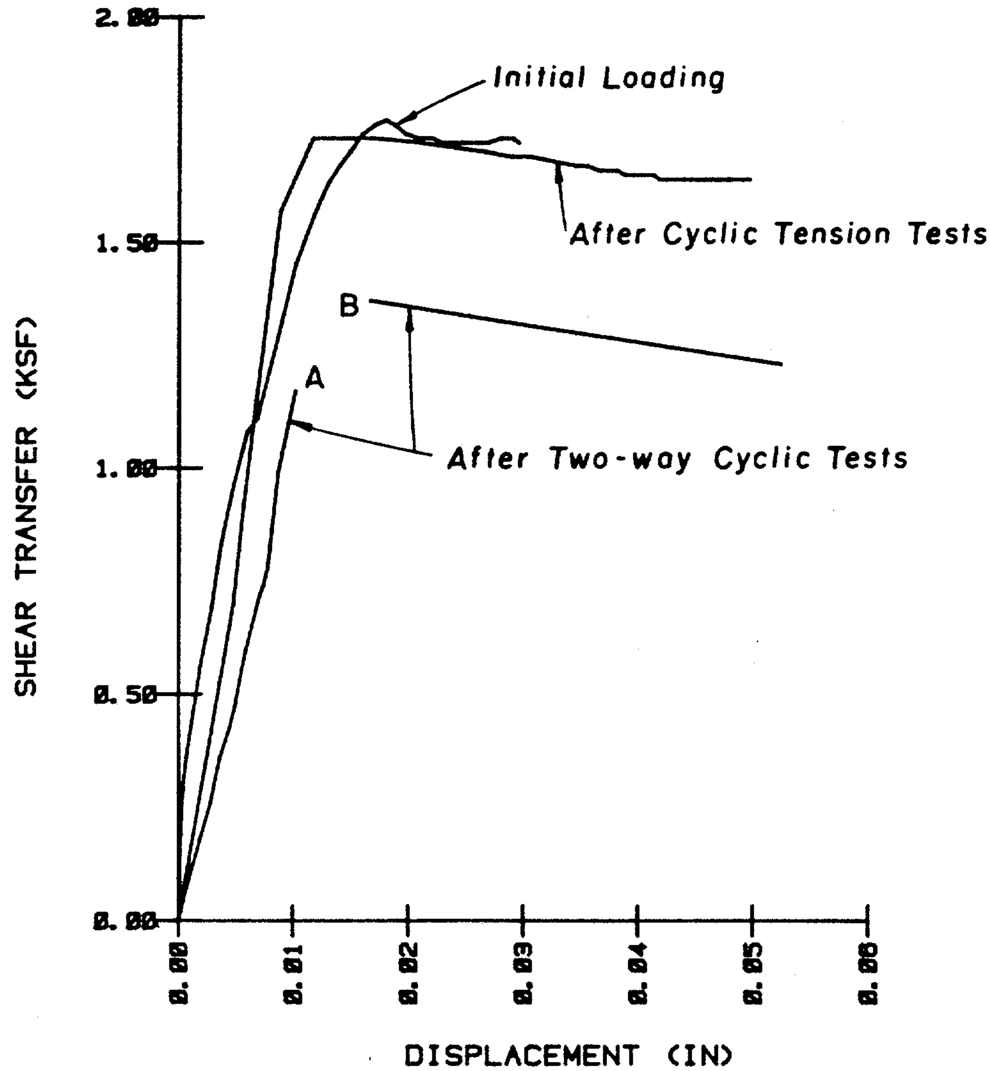
STRESS PATHS DURING LOAD TESTS AT THE 230-FT DEPTH
(X-PROBE)

The Earth Technology Corporation

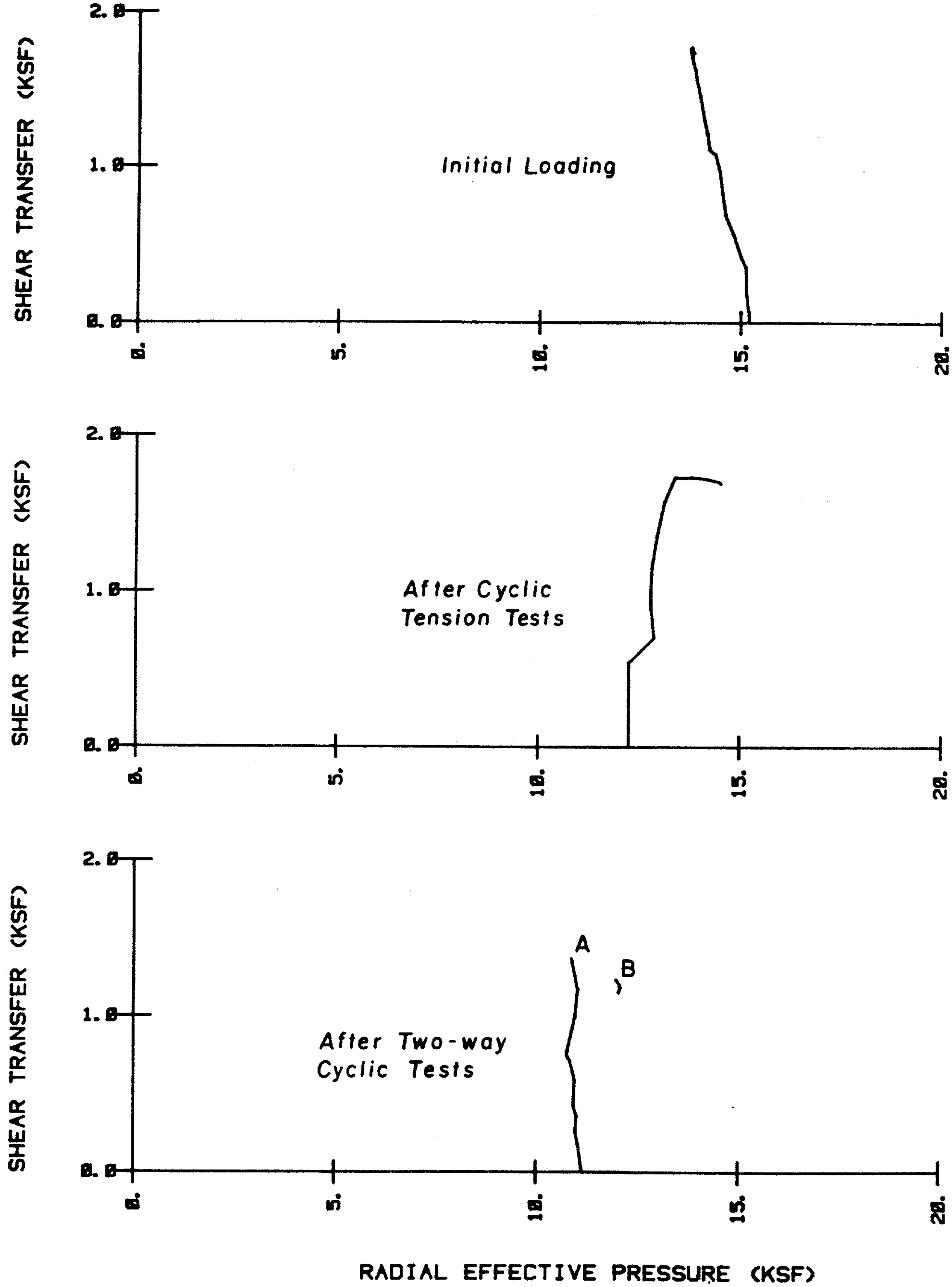


SOIL PRESSURES DURING CONSOLIDATION AT THE 250-FT DEPTH

The Earth Technology Corporation

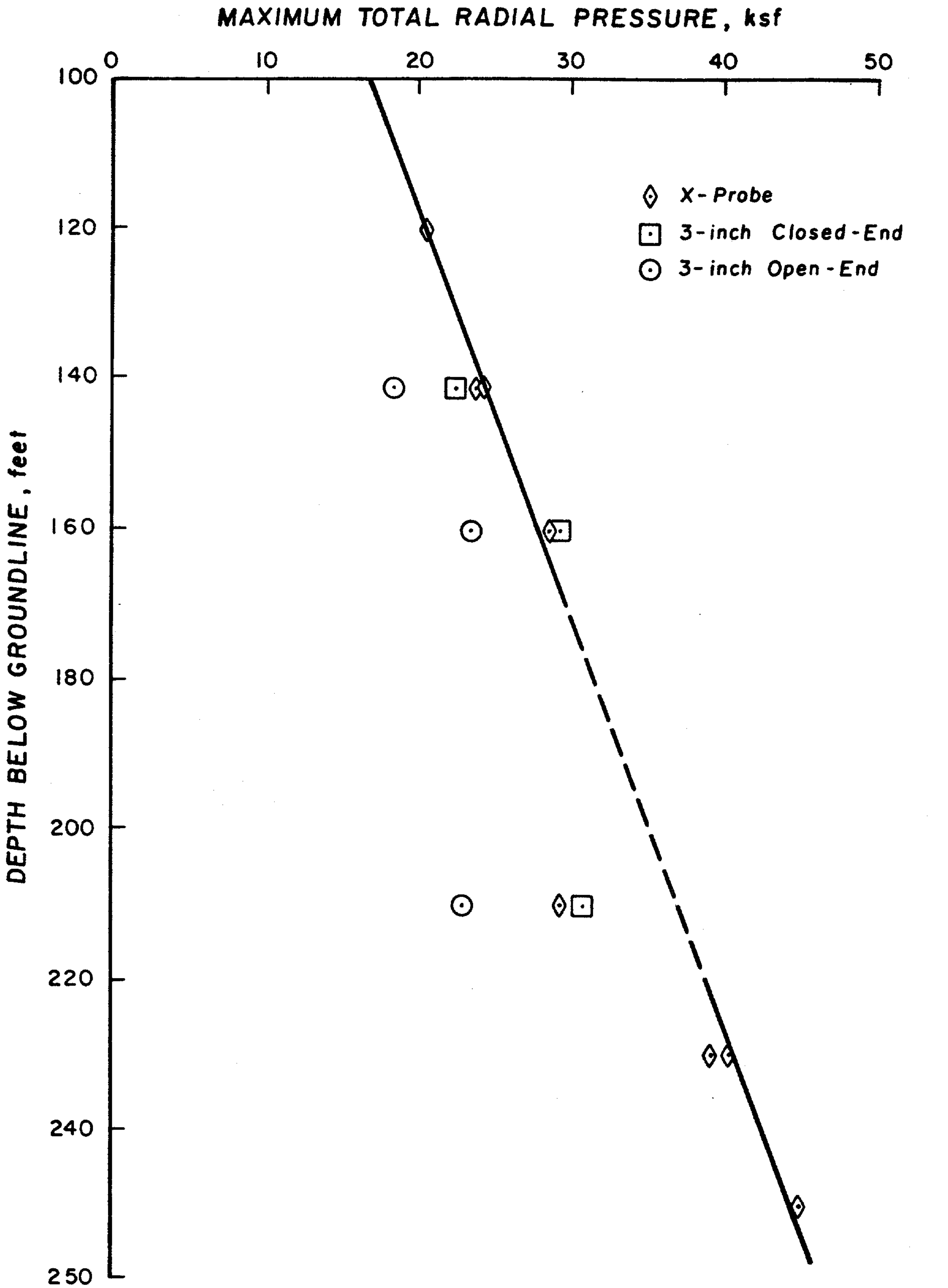


RESULTS OF THE CYCLIC TEST PROGRAM AT THE 250-FT DEPTH
(X-PROBE)



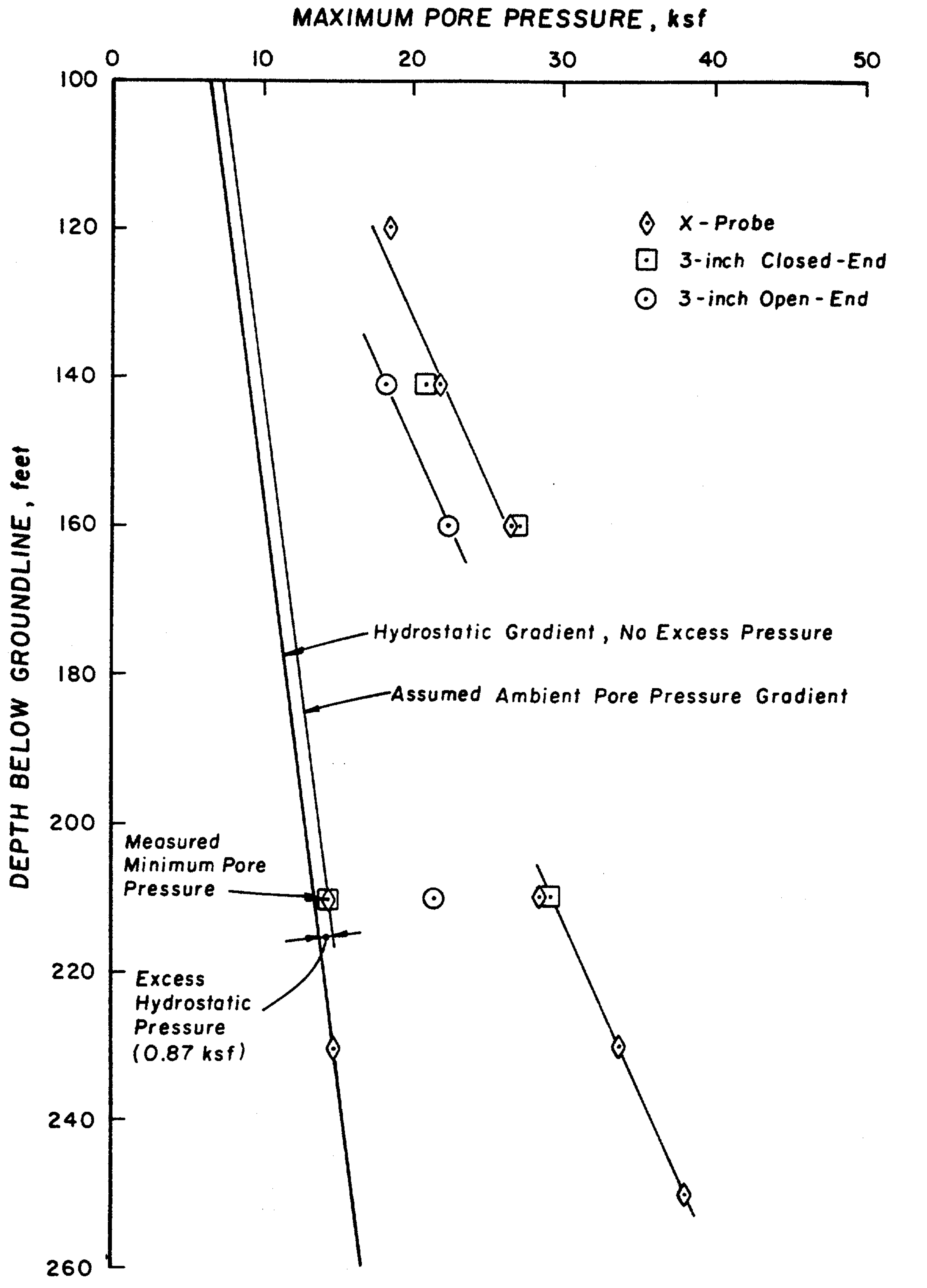
STRESS PATHS DURING LOAD TESTS AT THE 250-FT DEPTH
(X-PROBE)

The Earth Technology Corporation



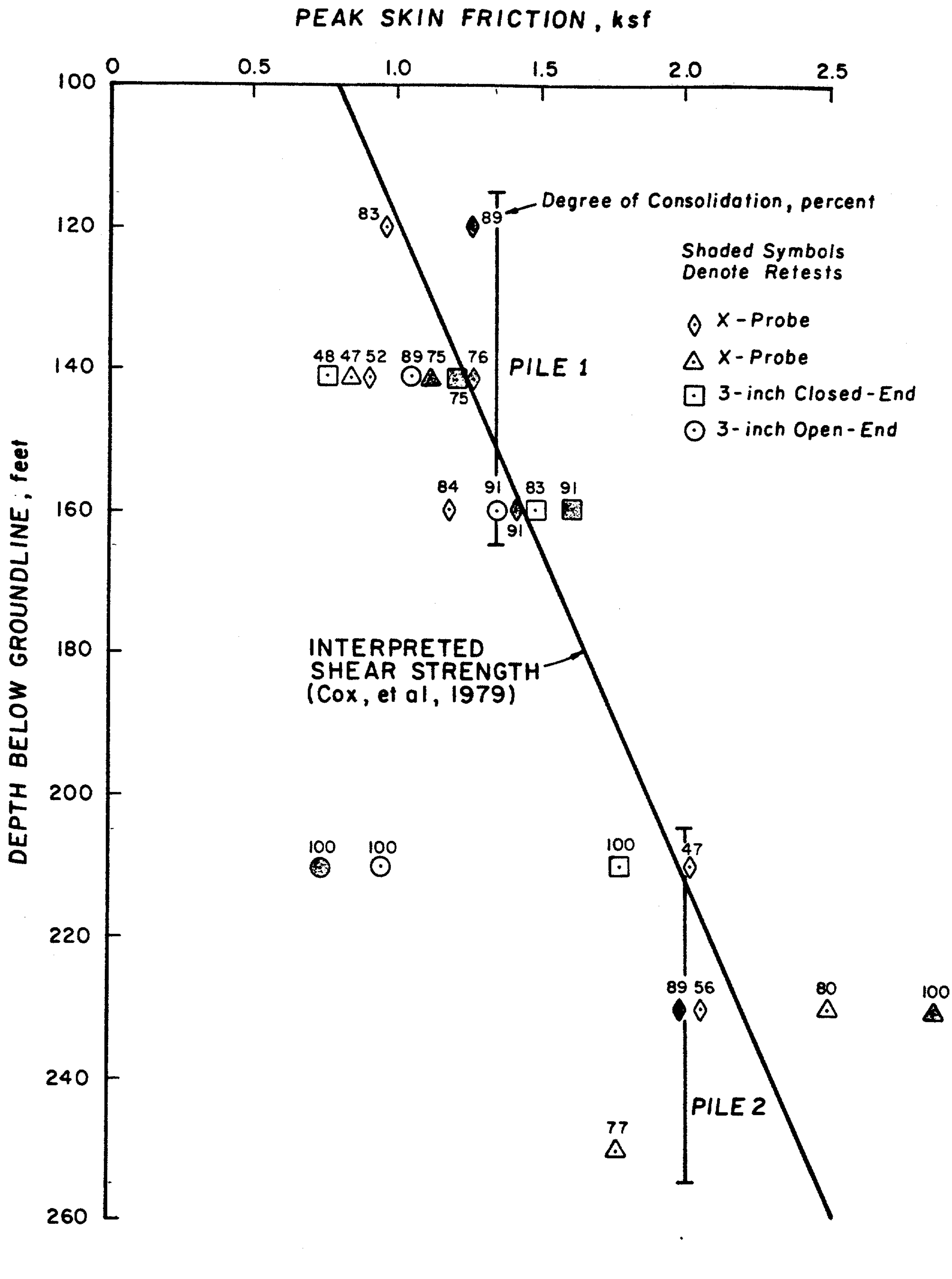
VARIATION OF MAXIMUM TOTAL PRESSURE WITH DEPTH

The Earth Technology Corporation



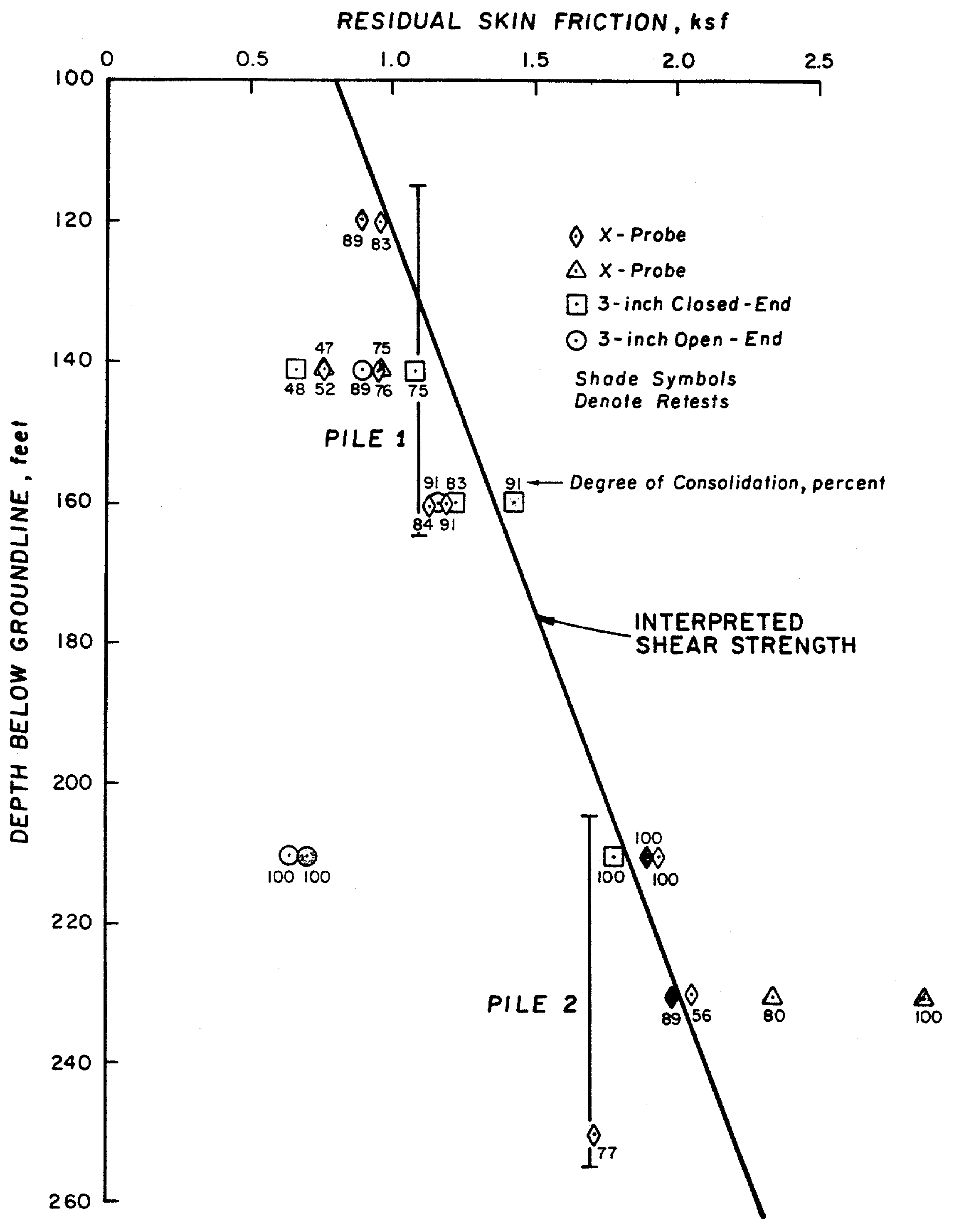
VARIATION OF MAXIMUM PORE PRESSURE WITH DEPTH

The Earth Technology Corporation

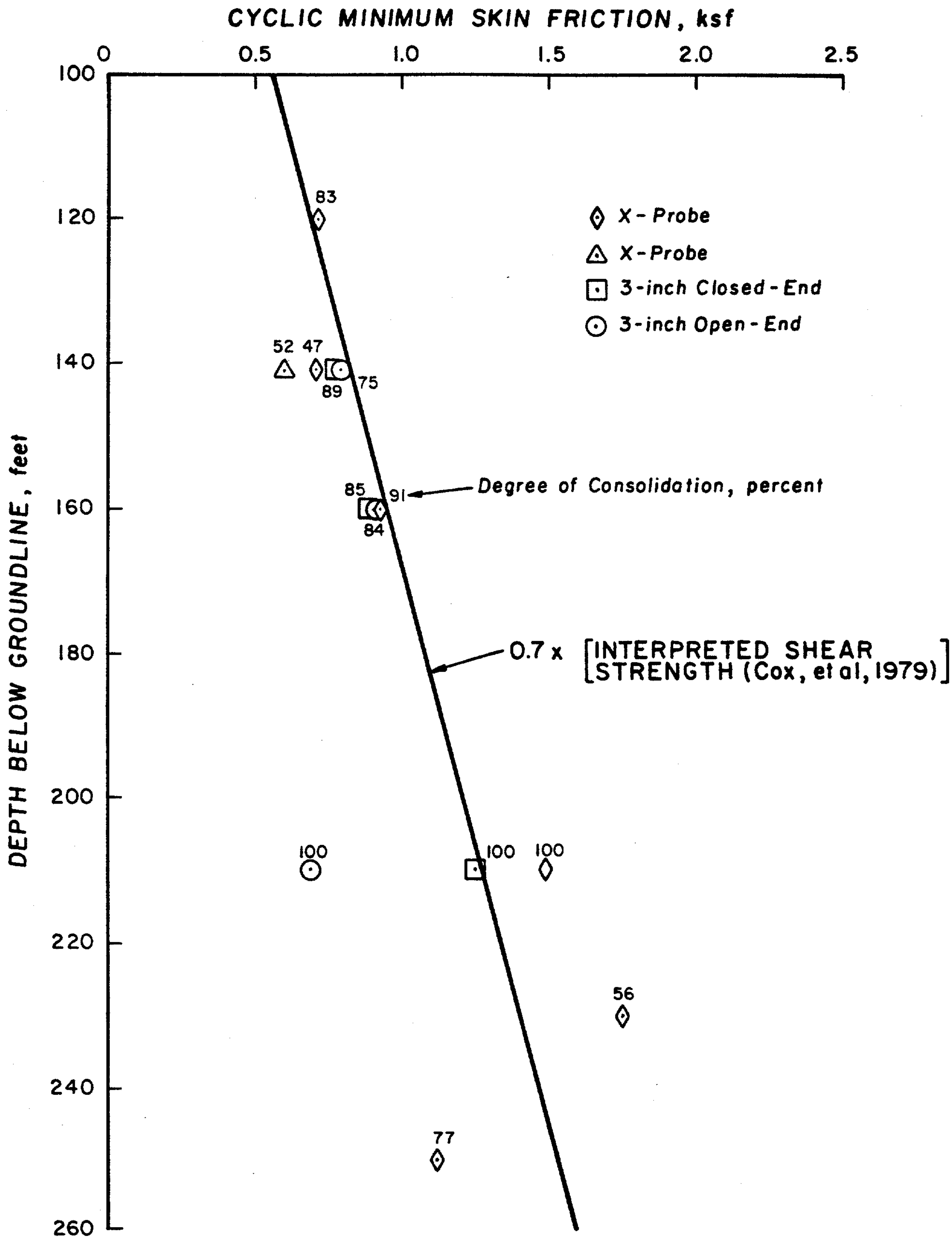


VARIATION OF PEAK SKIN FRICTION WITH DEPTH

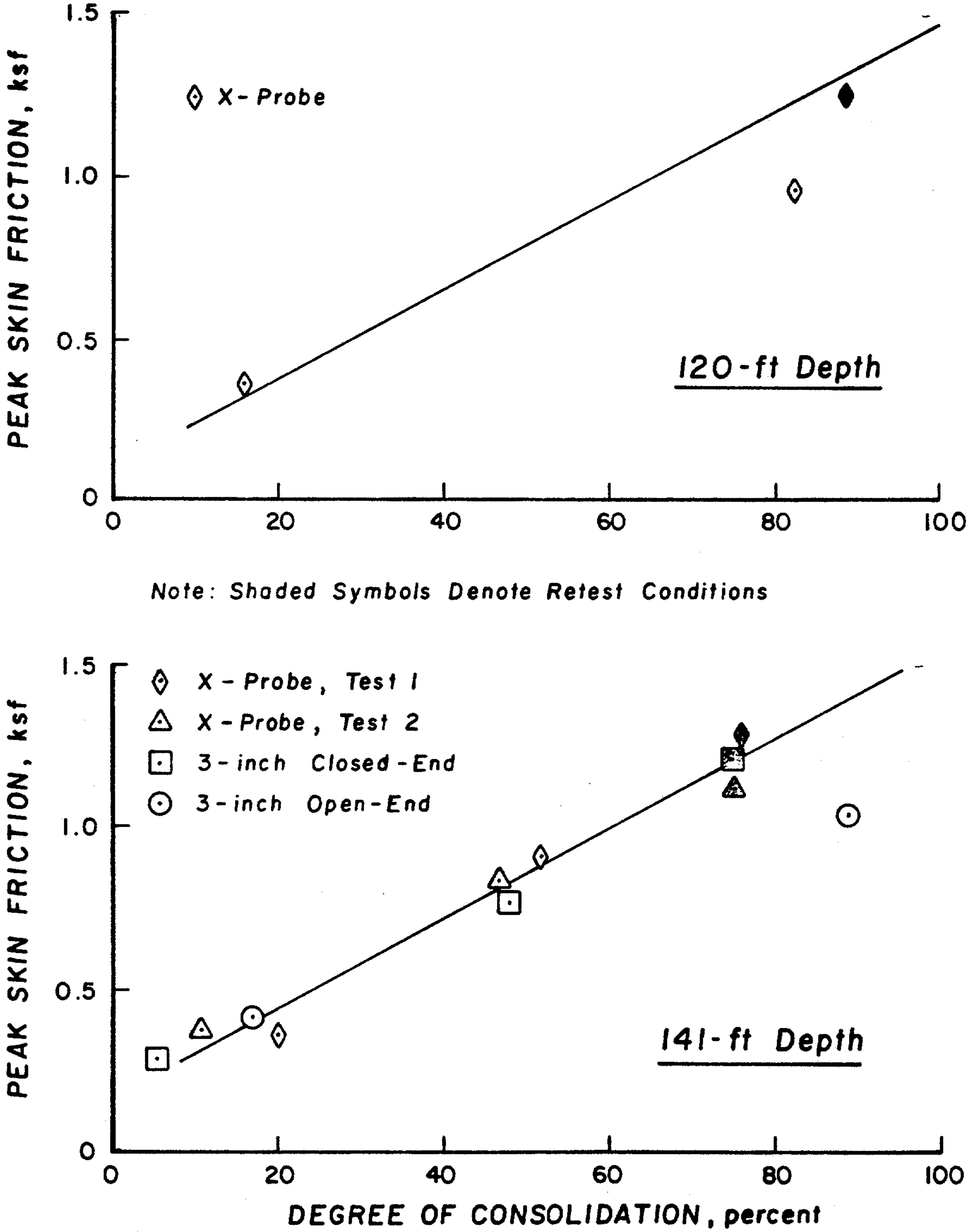
The Earth Technology Corporation



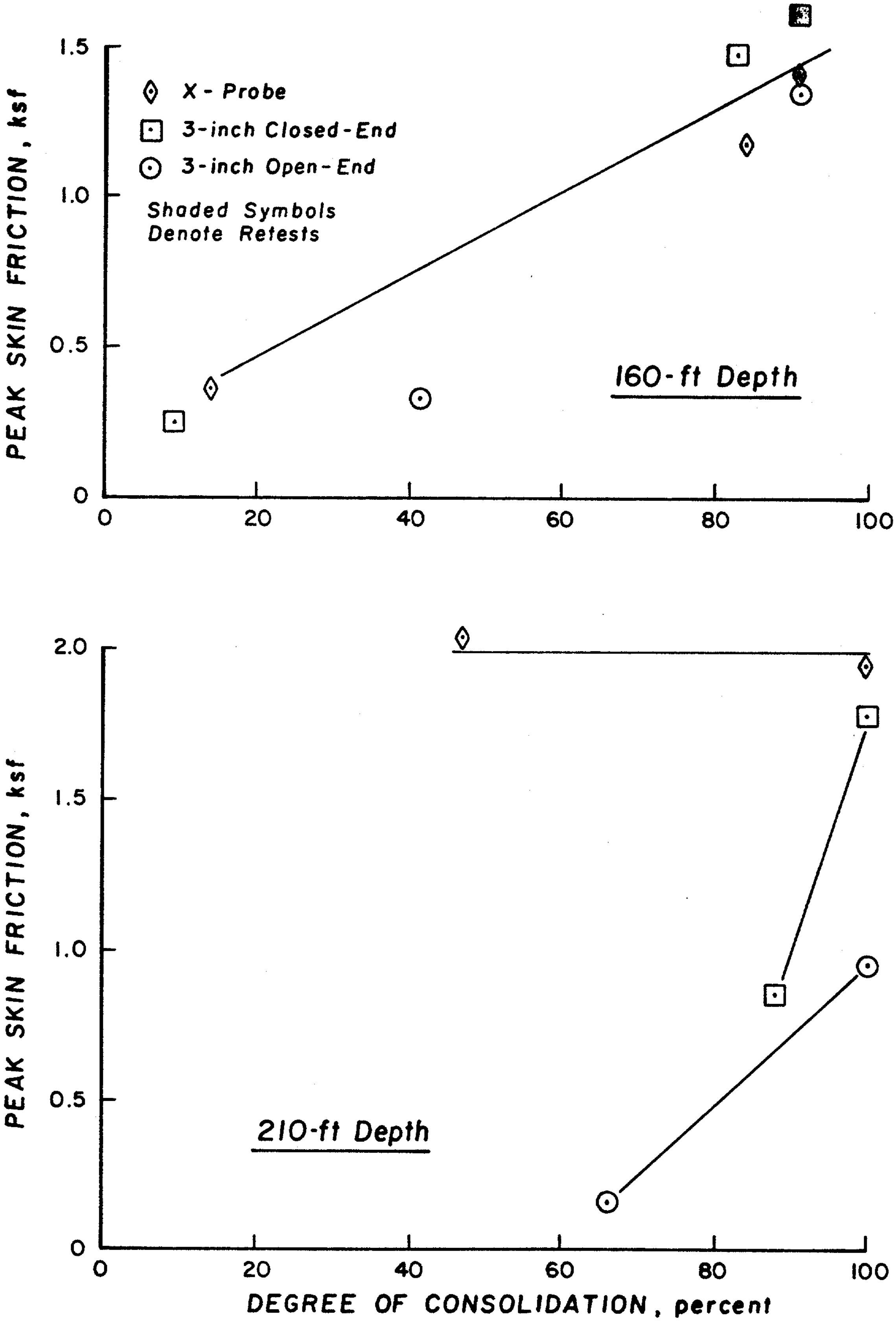
VARIATION OF RESIDUAL SKIN FRICTION WITH DEPTH



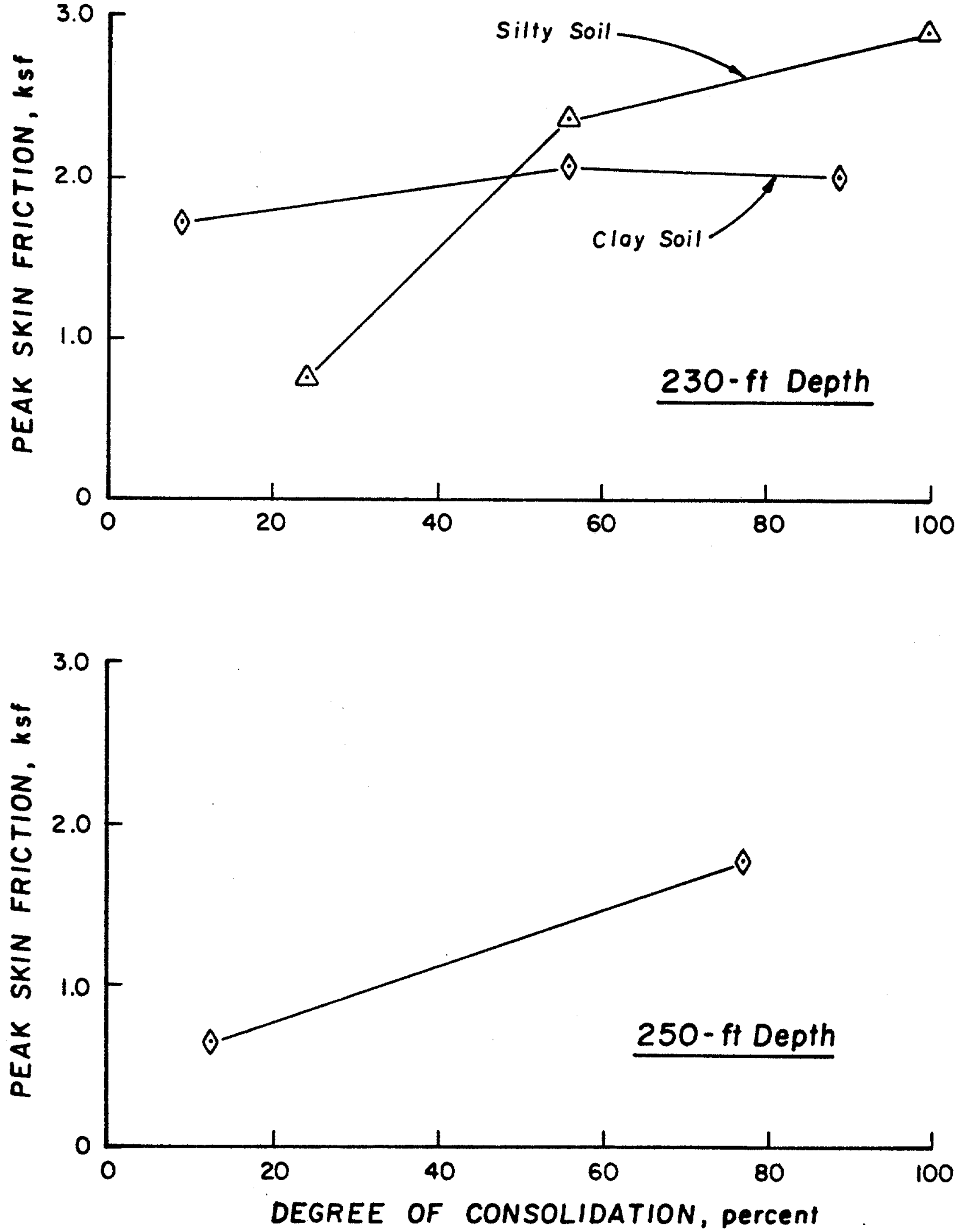
VARIATION OF CYCLIC MINIMUM SKIN FRICTION WITH DEPTH



VARIATION IN SKIN FRICTION WITH CONSOLIDATION

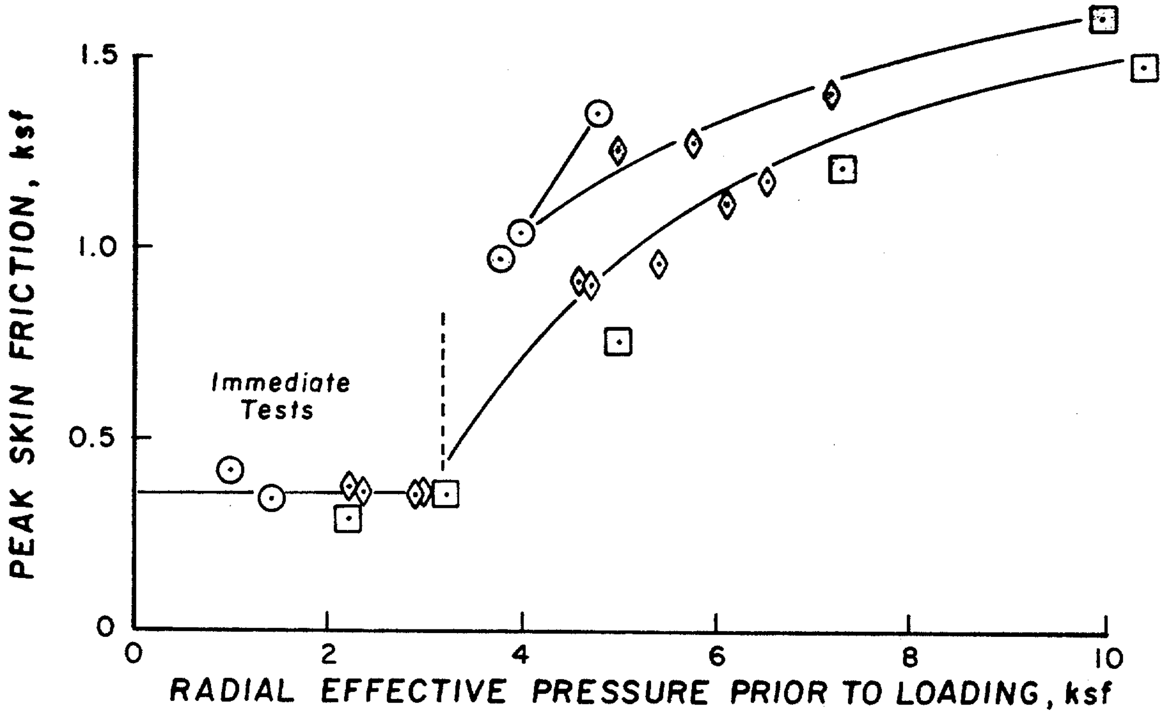


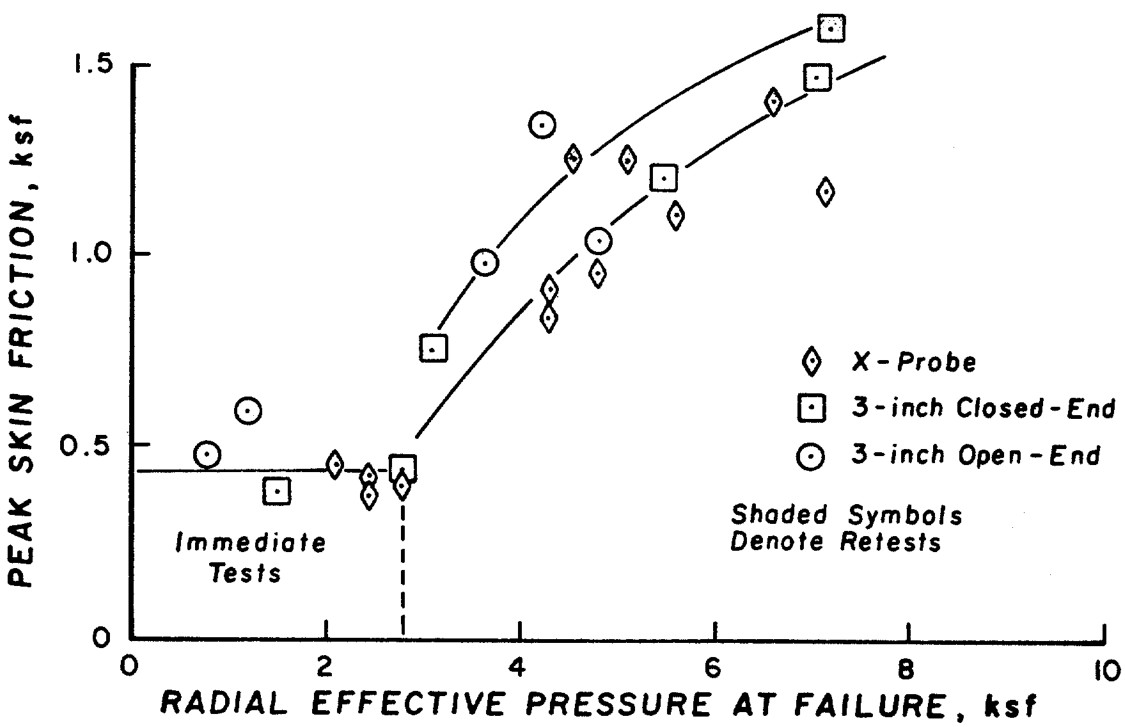
VARIATION IN SKIN FRICTION WITH CONSOLIDATION



VARIATION IN SKIN FRICTION WITH CONSOLIDATION

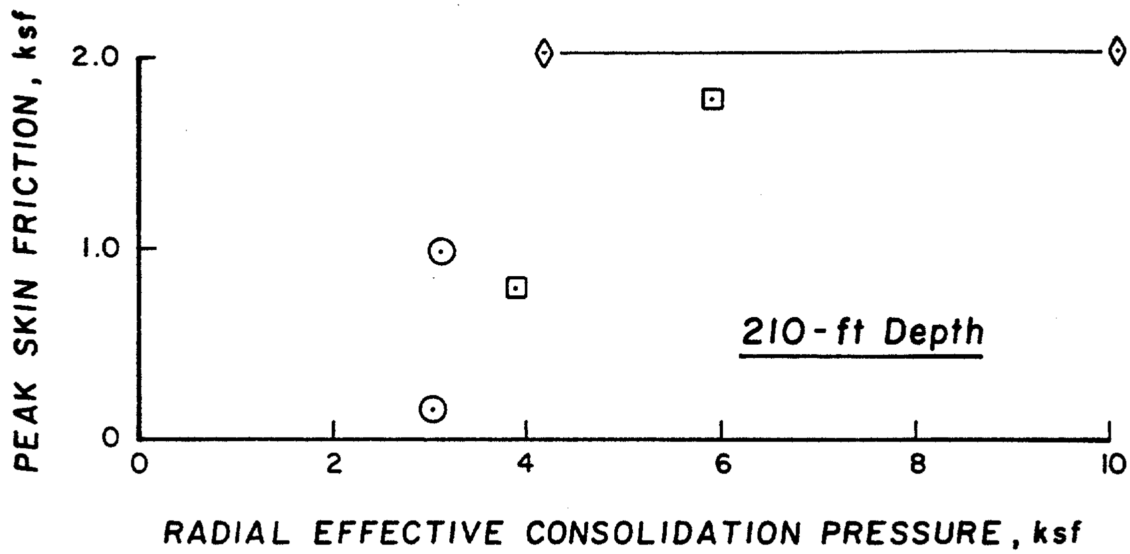
JOB NO. ____

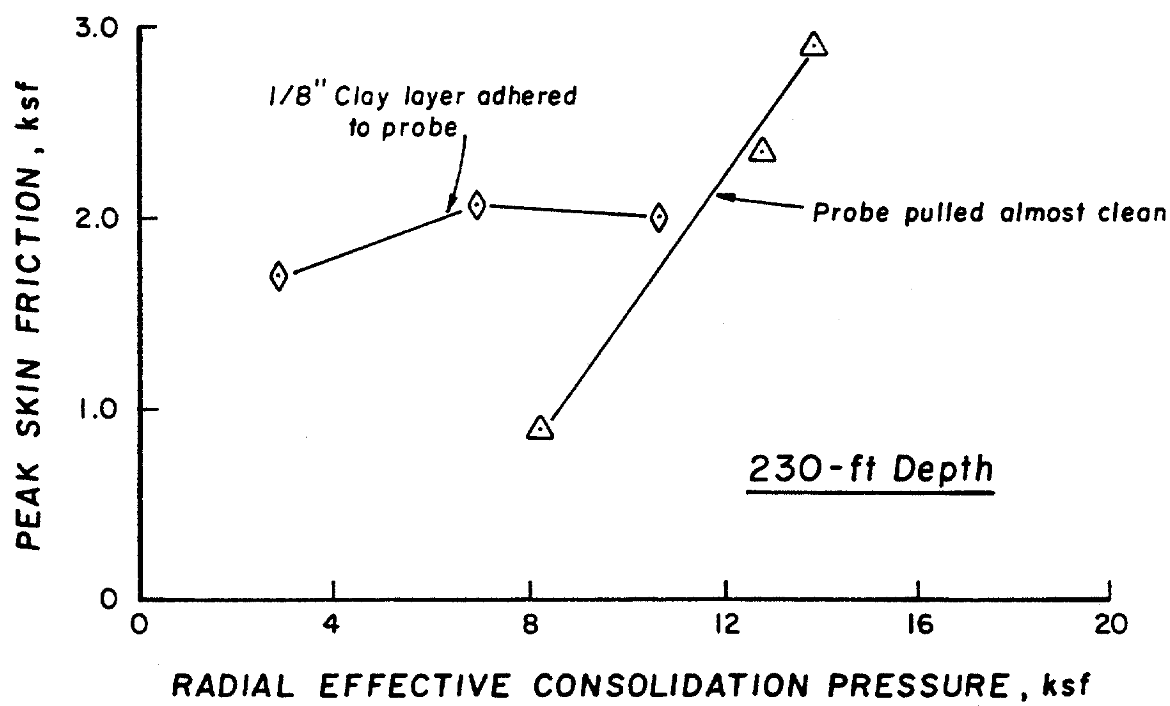




VARIATION IN SKIN FRICTION WITH
RADIAL EFFECTIVE PRESSURE
(120 to 160-foot DEPTHS)

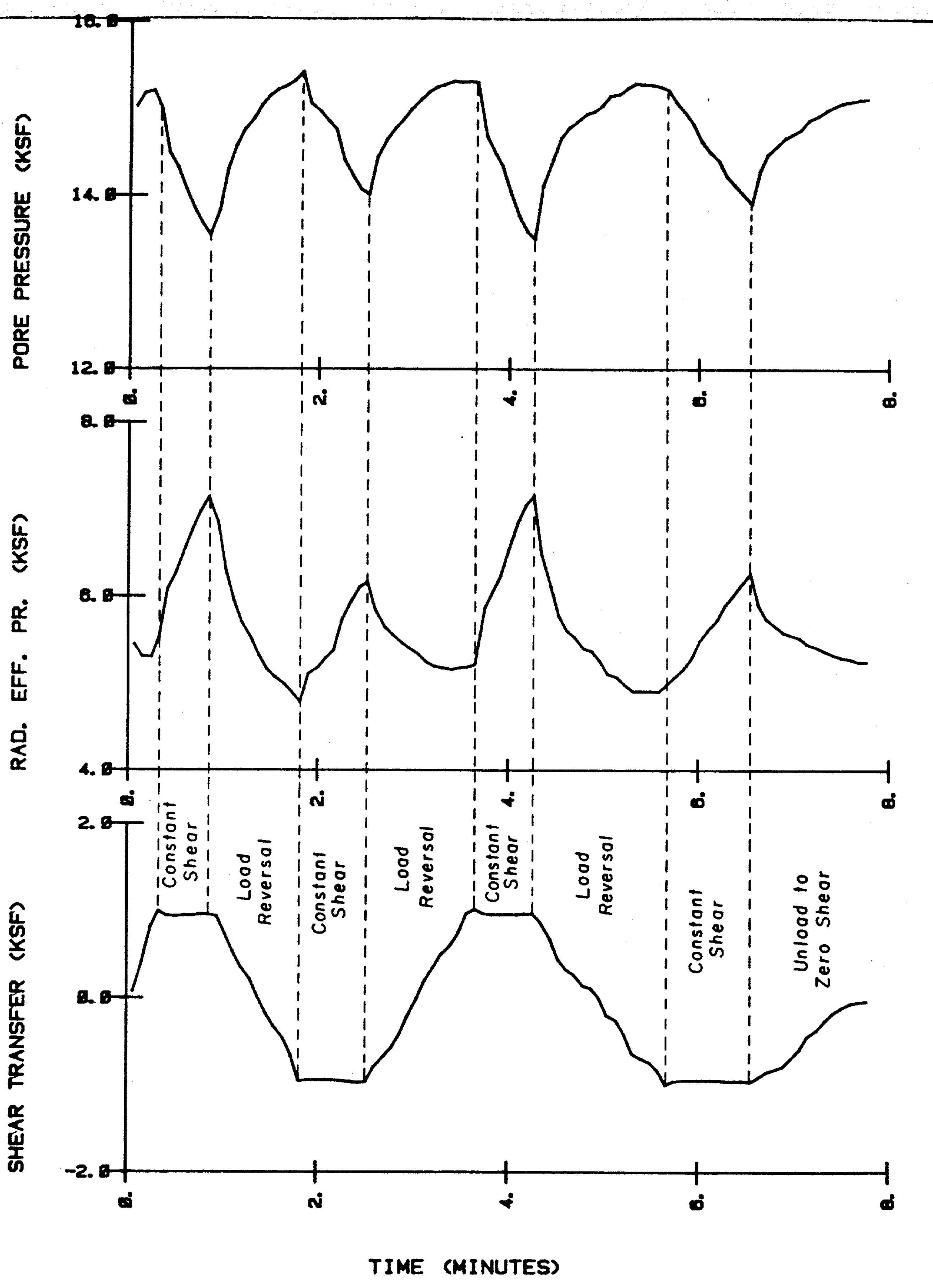
The Earth Technology Corporation





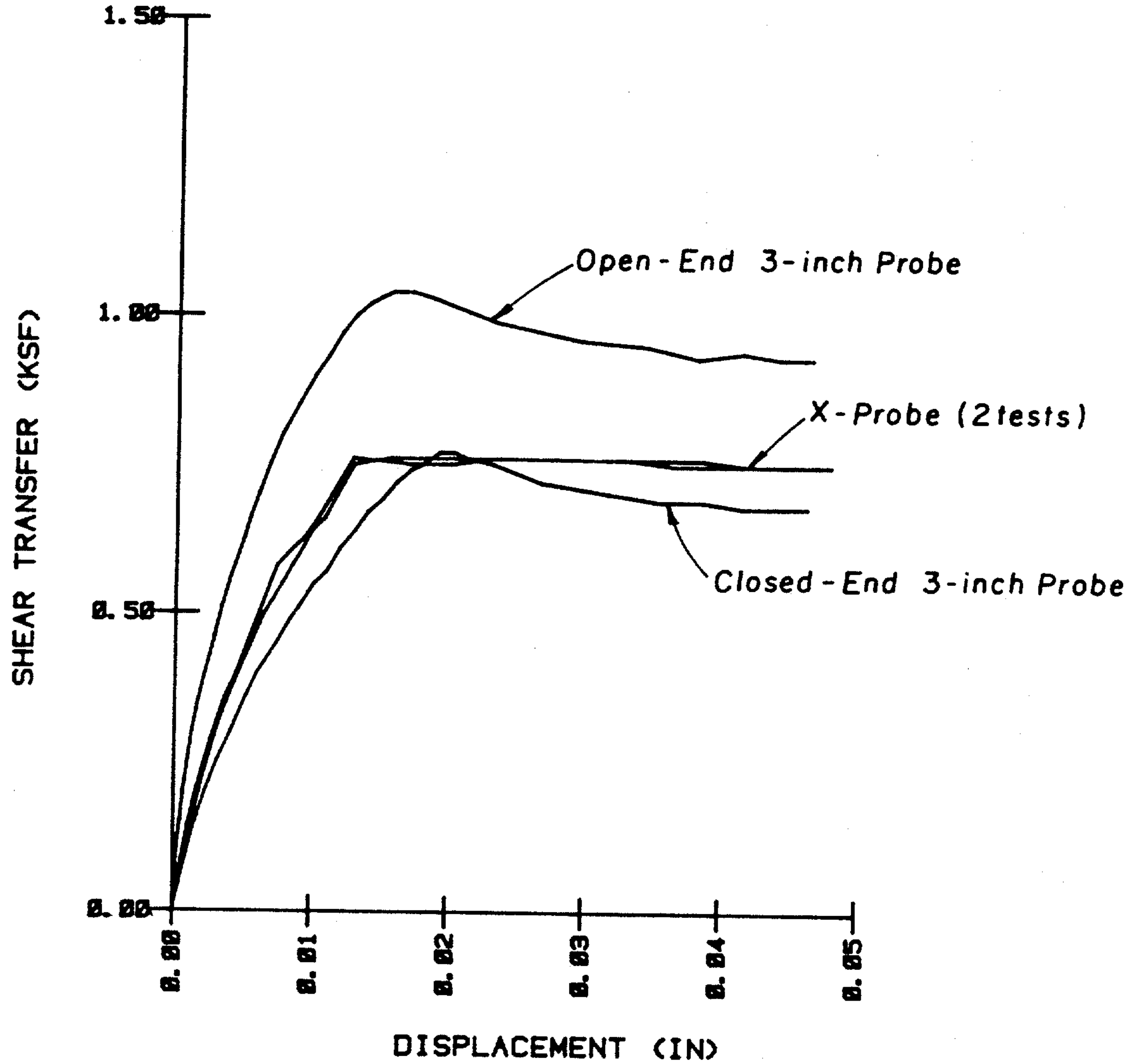
VARIATION IN SKIN FRICTION WITH CONSOLIDATION PRESSURE

The Earth Technology Corporation

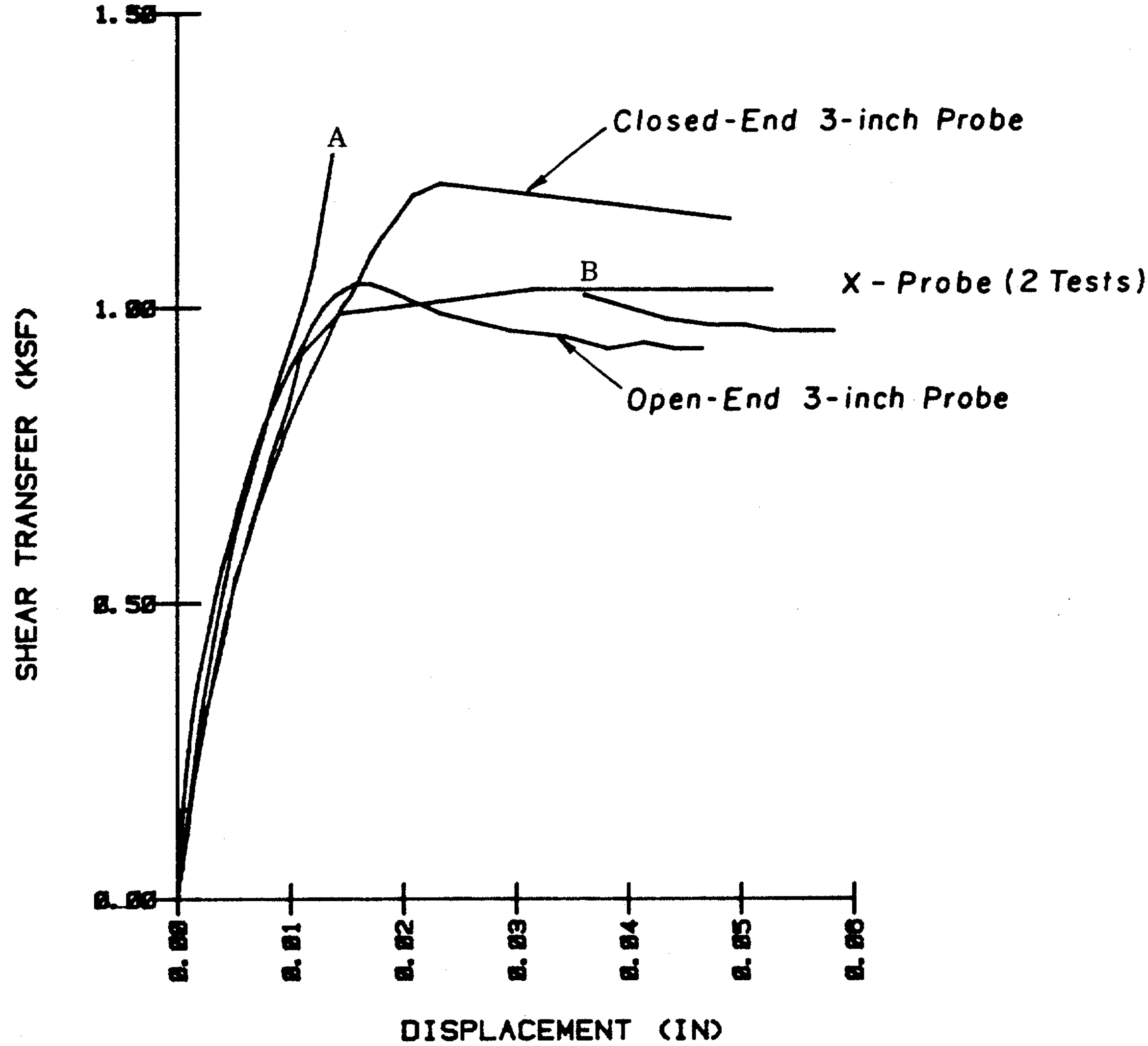


PRESSURE VARIATION DURING TWO-WAY CYCLIC TESTS
AT THE 160-FT DEPTH (X-PROBE)

The Earth Technology Corporation

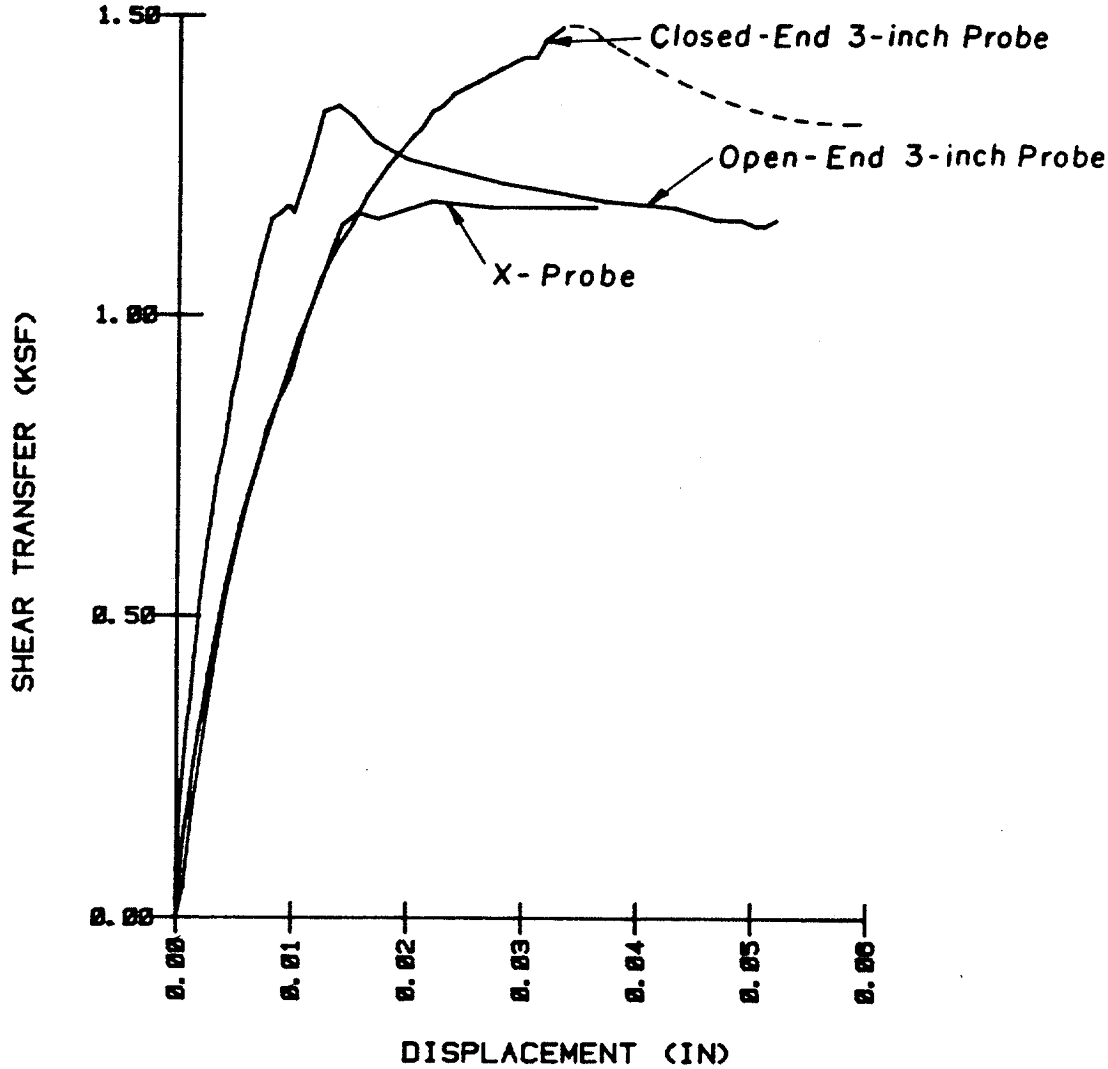


COMPARISON OF THE SHEAR-DISPLACEMENT BEHAVIOR AT THE 141-FT DEPTH DURING THE INITIAL LOADING

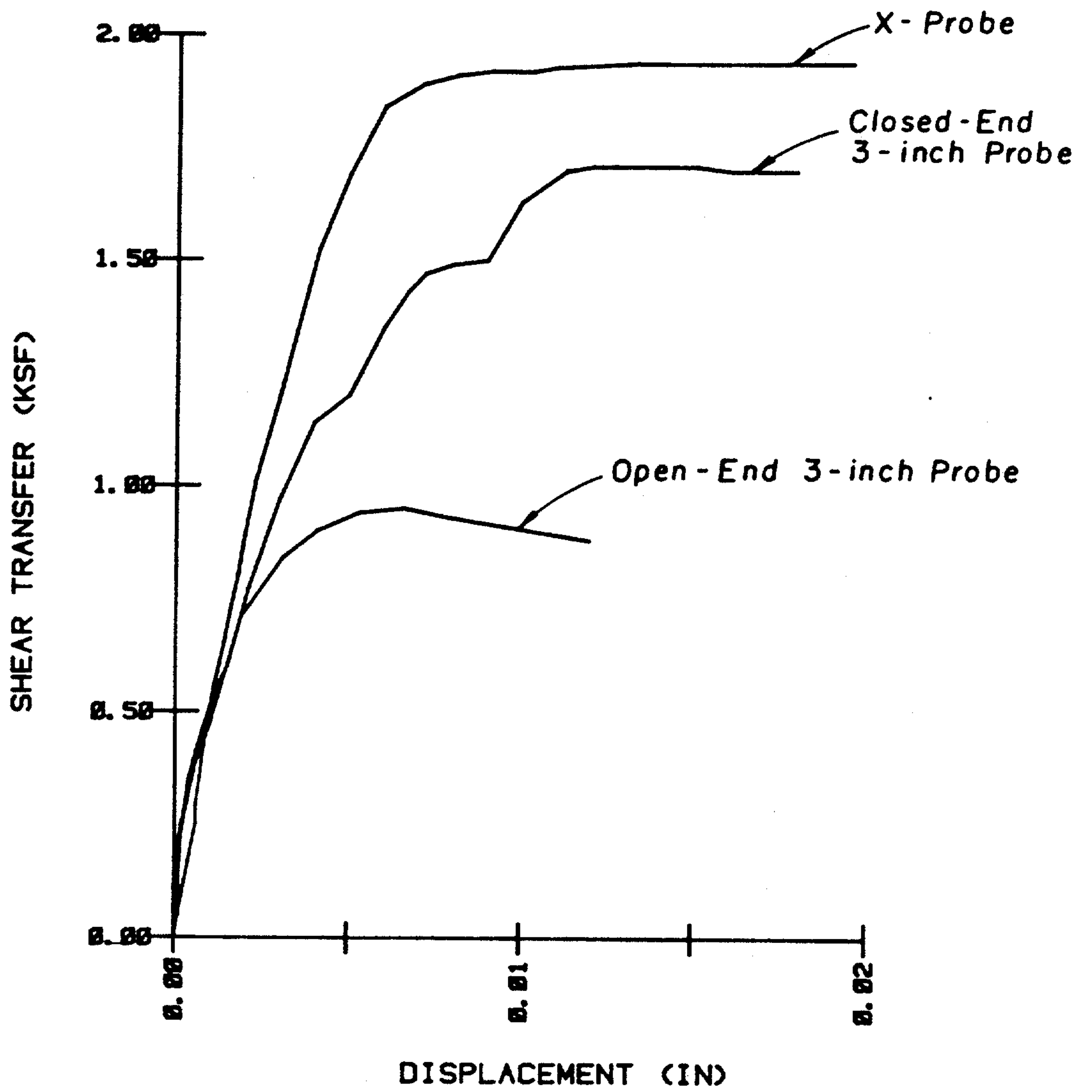


COMPARISON OF THE SHEAR-DISPLACEMENT BEHAVIOR AT THE 141-FT DEPTH AT SIMILAR DEGREES OF CONSOLIDATION

The Earth Technology Corporation

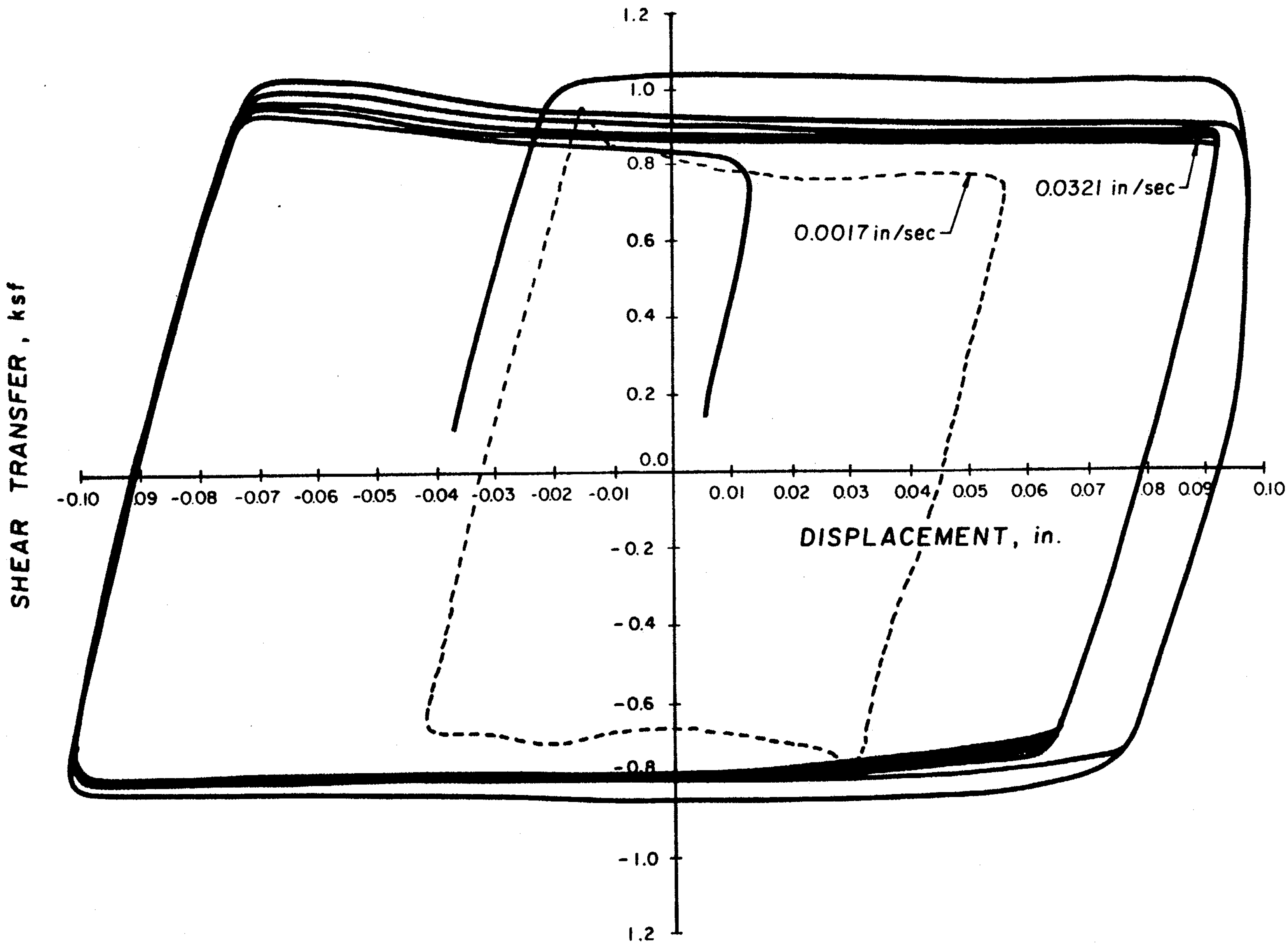


COMPARISON OF THE SHEAR-DISPLACEMENT BEHAVIOR
DURING INITIAL LOADING AT THE 160-FT DEPTH



COMPARISON OF THE SHEAR-DISPLACEMENT
BEHAVIOR AT THE 210-FT DEPTH

JOB NO. ______



EFFECTS OF RATE ON LIMITING SHEAR TRANSFER AT THE 120-FT DEPTH (X-PROBE)

SMALL-DIAMETER PILE SEGMENT TEST
AT EMPIRE, LOUISIANA
Volume II

By

The Earth Technology Corporation
Houston, Texas

August 31, 1984

THIS REPORT IS CONFIDENTIAL

The contents of this report are proprietary in nature and shall remain confidential for a period of three (3) years following the date of issue of this Report.

TABLE OF CONTENTS

Volume II

INTRODUCTION

This report contains the results of a series of experiments performed at the site of four load tests on 14-in. diameter piles. The experiments were performed with instrumented pile-segment models having diameters of 1.72 and 3.00 in. The models were installed at six depths in four borings. Two sets of three depths each were selected so that the experiments would be performed at depths corresponding to the top, middle, and bottom portions of the embedded lengths of two of the 14-in. diameter pile tests.

The initial volume of this report contains the analysis and interpretation of the results of the experiments. This volume contains the detailed results of all the experiments. Portions of the data contained herein are given in Volume I. This volume contains a complete record of the experiments.

The data taken during each experiment at each depth in each boring are presented separately. The plates in each appendix contain all the data which were recorded during the course of an experiment in the chronological order in which the data were recorded. The plates are thus arranged in the order in which the load tests were performed, following the general set of guidelines given below. As reported in Volume I, the procedures were followed closely in the 120, 141, and 160-ft depths, with only two exceptions: the 3-in. open-end probe at 141 ft, and the second X-probe experiment at the 141-ft dpeth. In the lower layers, the procedures were changed because of the nature of the behavior recorded during the experiments. Had the layer been a uniform clay, deviations from the procedures would not have been necessary.

The procedures followed during each experiment were:

1. Install the probes either by driving (3-in. probes) or by pushing (X-probe).

2. Perform a test to failure, in both tension and compression, as soon after installation as possible.

3. Observe the dissipation of the exces pore pressure. Allow the process to proceed for a predetermined length of time for each experiment.

4. Perform a slow monotonic loading to failure in tension.

5. Allow any excess pore pressure generated by the test to return to the value recorded prior to the test.

6. Perform a sequence of one-way cyclic (repeated) tension tests, with a bias load of 50 percent of the failure load from Step 4, and with a cyclic component above and below the bias being progressively increased in increments of 10 percent, until failure is indicated by progressive upward displacement of the probe on each application of the maximum load.

7. Perform a slow monotonic loading to failure in tension.

8. Perform a two-way, displacement-controlled, cyclic test, in order to obtain the maximum degree of cyclic degradation.

9. Increase the rate of loading to the maximum possible, and continue the two-way cyclic test until the values of plastic shear have stabilized.

10. Decrease the rate of loading to the original slow rate, and continue the two-way cyclic tests until the shear again stabilizes.

11. Stop the active loading, and monitor the dissipation of excess pore pressures for as long as possible, based on the time of installation of the probe for the next experiment.

12. Perform a slow monotonic loading to failure, both in tension and compression, immediately prior to removing the probe.

Only the results of the first experiment will be discussed in depth; the remaining experiments follow a similar pattern. It is felt that, by reviewing the two volumes in parallel, a more complete understanding of the experiments may be accomplished. It is therefore not necessary, nor desirable, to provide a redundant explanation or interpretation of all the results of each experiment individually.

APPENDIX A: SUMMARY OF FIELD ACTIVITIES

EMPIRE PROJECT - 84-002
SUMMARY OF FIELD ACTIVITIES

Date	From	To	Description of Activities
April 4			Demobilizing from an offshore project at Venice, La.
April 5	0800	1000	Cleaning the site, putting shells on the site.
	1000	1300	Mobilizing from Venice, La., unloading equipment and equipment shed at the site.
	1300	1600	Hooking up electrical power at the site.
	1600	1800	Obtaining building permit. Informed by Buras Parish permits office that we need to have a building permit in order to hook up electricity by LP&L.
April 6	0800	1300	Obtaining building permit. Capazzoli drilling crew arrived at the site at about 1200 hr.
	1300	1400	Waiting for LP&L to come.
	1400	1600	Hooking up electricity by LP&L.
	1600	1700	Preparing the site. Left the site at 1700 hr. Informed that we need to change boring locations.
April 7	0700	1100	Rearranging boring locations. Moved pipe racks and equipment shed to be near the borings.
	1100	1330	Setting up ground anchors. Drill crew arrived at the site 1100 hr.
	1330	1630	Drilling test hole to 128-ft penetration. Drill crew left the site at 1630 hr.
	1630	1800	Waiting for a load plate to arrive.
April 8	0700	1410	Drilling the test borings. Drill crew left the site at 1410 hr.
	1410	1800	Standing by at the site.
April 9	0500	1915	Trying to recover a load plate from an offshore platform. Standing by at the site. Got the load plate from offshore at 1915 hr.

Date	From	To	Description of Activities
April 10	0700	1015	Drilling and casing Hole No. 2 to the 141-ft test penetration.
	1015	1120	Installing open-end 3-in. tool in Hole No. 2.
	1120	1200	Setting up the loading system.
	1200	1300	Performing the immediate test at 141-ft penetration (Hole No. 2)
	1300	1730	Drilling and casing Hole No. 3 to 141-ft test penetration.
	1730	1815	Moving ram and loading frame. Drill crew left the site at 1815 hr.
	1815	1840	Checking X-probe. Had noise problem with the instruments.
	1840	2300	Staying at the motel. Arrived at the site at 2300 hr.
	2300	2400	Checking X-probe again.
April 11	0000	0100	Checking X-probe. Still having noise problem with the instruments.
	0100	0410	Performing the 12-hr test in Hole No. 2 at test penetration 141 ft (3-in. tool, open-end)
	0410	0600	Staying at the motel.
	0600	0700	Picking up a second loading plate at Venice (shipping from Houston).
	0700	1100	Drilling crew arrived at the site at 0700 hr. Standing by due to the noise problem with the instruments.
	1100	1530	Electrician is checking the power switch box. The problem is related to the ground connections. J. Audibert arrived at the site at 1200 hr. Drillers left the site at 1530 hr.
	1700	2000	Staying at the motel.
	2000	2345	Picking up N. Dwyer at the MSY. Arrived at Empire at 2345 hr.

Date	From	To	Description of Activities
April 12	0700	0900	Re-connecting the ground wire inside power switch box. No more noise problems. Checking the X-probe. Drillers arrived at the site at 0815 hr.
	0900	0930	Stringing the X-probe cable into N-rods.
	0930	1015	Checking the closed-end 3-in. tool in Hole No. 3.
	1015	1140	Installing the 3-in. tool (closed-end) into Hole No. 3 at 141-ft test penetration.
	1140	1220	Performing the immediate test in Hole No. 3.
	1220	1300	Lunch break for Earth Technology crew.
	1300	1400	Pulling N-rods from Hole No. 2.
	1400	1600	Drilling and casing to 160-ft test penetration in Hole No. 2.
	1600	1640	Installing 3-in. tool (open-end) into Hole No. 2 at 160-ft test penetration. JMEA, SRB and ND left for Houston at 1600 hr.
	1640	1650	Driving the 3-in. tool to the desired penetration. wire in cable parted during driving. Abort the test.
	1650	1730	Pulling the 3-in. tool from Hole No. 2. Drill crew left the site at 1730 hr.
	1730	1800	Checking the 3-in. tool. Left the site at 1800 hr.
	1800	2340	Staying at motel.
	2340	2400	Performing the 12-hr test in Hole No. 3 on 3-in. tool (closed-end) at 141-ft test penetration. Arrived at the site at 2340 hr.
April 13	0000	0500	Performing the 12-hr test in Hole No. 3 at 141-ft test penetration. Ground anchors (four) did not hold during compression loading. Left site at 0500 hr.
	0500	0700	Staying at motel.
	0700	1500	Drilling and casing to 120-ft penetration at Hole No. 1. Buying 12 more ground anchors from Deltone. Received them at 1930 hr.
	1500	1700	Checking X-probe at Hole No. 1.

Date	From	To	Description of Activities
April 13	1700 -	1740	Installing X-probe at Hole No. 1 to 120-ft test penetration. Lost output from friction sleeve during the push of X-probe. Abort the test.
	1740 -	1815	Checking X-probe. Left the site at 1815 hr.
April 14	0700 -	0830	Pulling X-probe from Hole No. 1.
	0830 -	1100	Checking the instruments. Setting anchors: 0915 - 1100 hr.
	1100 -	1215	Drilling and casing in Hole No. 1
	1215 -	1245	Lunch break.
	1245 -	1340	Moving ram and loading frame to Hole No. 3. Setting up 8 anchors.
	1340 -	1630	Drilling and casing in Hole No. 2.
	1630 -	1810	Testing the close-end 3-in. close-end tool in Hole No. 3 at 141-ft test penetration (with reinforced 8 ground anchors). Drill crew left site at 1715 hr. Left site at 1810 hr.
April 15	0700 -	0810	Moving ram and loading frame away from Hole No. 3.
	0810 -	0930	Drilling and casing in Hole No. 2.
	0930 -	1005	Installing the 3-in. tool (open-end) in Hole No. 2. Had problems with the tool.
	1005 -	1100	Pulling N-rods and 3-in. tool from Hole No. 3.
	1100 -	1400	Drilling and casing in Hole No. 3 to 160-ft test penetration.
	1400 -	1450	Installing 3-in. tool (closed-end) into Hole No. 3.
	1450 -	1530	Driving 3-in. closed-end tool to 160-ft penetration at Hole No. 3. Setting up loading frame and anchors.
	1530 -	1545	Performing the immediate test.
	1545 -	1645	Stringing the cable into N-rods. Drill crew left the site at 1645 hr.
	1645 -	1830	Working on X-probe tool. Left site at 1830 hr.

Date	From	To	Description of Activities
April 16	0700	0930	Working on 3-in. tools.
	0930	1010	Checking 3-in. tools in Hole No. 2. B. Prindle (Sandia) arrived at the site at 1000 hr.
	1010	1055	Installing 3-in. tool into Hole No. 2.
	1055	1105	Driving the 3-in. tool to 210-ft test penetration in Hole No. 2.
	1105	1124	Performing the immediate test.
	1124	1200	Lunch break.
	1200	1250	Spooling the bad cable.
	1250	1450	Drilling to 141-ft test penetration at Hole No. 1.
	1450	1550	Installing the X-probe in Hole No. 1.
	1550	1600	Performing the immediate test on X-probe in Hole No. 1 at 141-ft test penetration.
	1600	1640	Monitoring the pore pressure dissipation. Left the site at 1640 hr.
	1640	1930	Staying at motel.
	1930	2030	Preparing the 4 -hr test on X-probe in Hole No. 1. Arrived at the site @ 1930 hr.
	2030	2400	Performing the 4-hr test on X-probe in Hole No. 1 at 141-ft penetration.
April 17	0000	0030	Performing the 4-hr test. Left the site at 0030 hr.
	0700	0812	Performing pullout test in Hole No. 1 at 141-ft test penetration.
	0812	0905	Pulling X-probe from Hole No. 1.
	0905	1050	Drilling and casing to 160-ft test penetration in Hole No. 1.
	1050	1130	Installing X-probe to 160-ft test penetration in Hole No. 1.
	1130	1200	Performing the immediate test. Drill crew left the site at 1200 hr.

Date	From To	Description of Activities
April 17	1200 - 1300	Lunch break.
	1300 - 1400	Working on instruments at the site.
	1400 - 1700	Standing by at motel.
April 18	0800 - 1100	Preparing for the test. Left for MSY to pick up a tool at 0800 hr.
	1100 - 1540	Performing 24-hr test in Hole No. 1 on X-probe tool at 160-ft test penetration.
	1540 - 2130	Performing 72-hr test in Hole No. 3 on closed-end 3-in. tool at 160-ft test penetration. Left the site at 2130 hr.
April 19	0700 - 0750	Performing pull-out test on X-probe in Hole No. 1 (160-ft penetration).
	0750 - 0900	Moving ram and pulling X-probe from Hole No. 1.
	0900 - 1200	Drilling and casing to 210-ft test penetration. Took soil sample at 205-207 ft.
	1200 - 1600	Performing 72-hr test at Hole No. 2, open-end 3-in. tool at 210-ft penetration.
	1600 - 1710	Installing X-probe in Hole No. 1 to 210-ft test penetration.
	1710 - 1730	Performing the immediate test.
	1730 - 1830	Pulling out 8 anchors from Hole No. 2. Drill crew left the site at 1800 hr.
April 20	0700 - 0740	Performing pullout test on 3-in. tool, Hole No. 2 at 210-ft penetration.
	0740 - 0800	Moving ram to Hole No. 1. Got ready for 24-hr test on X-probe at 210-ft penetration.
	0800 - 0850	Pulling 3-in. tool and N-rods from Hole No. 2.
	0850 - 1130	Performing X-probe test at 210-ft penetration in Hole No. 1. Pulling 4-in. casing from Hole No. 2 at 0850 hr - 1100 hr. Setting anchors at 1100-1130 hr.

Date	From To	Description of Activities
April 20	1130 - 1140	Setting up Hole No. 4.
	1140 - 1600	Drilling and casing Hole No. 4 to 120-ft test penetration. Took a soil sample at 108-110 ft to confirm the soil stratigraphy.
	1600 - 1650	Pulling X-probe and N-rods from Hole No. 1. HM arrived at the site at around 1600 hr.
	1650 - 1800	Checking and installing X-probe into Hole No. 4. at 120-ft test penetration.
	1800 - 1830	Performing the immediate test in Hole No. 4. Left the site at 1830 hr.
April 21	0700 - 0730	Preparing at the site. Drill Crew arrived at 0700 hr. A power pole fell on the roof of white building at 0730 hr. HM arrived at the site at 0729 hr.
	0730 - 0900	Emergency repairs on the power pole problem. HM was in charge of the remedial measures.
	0900 - 0950	Pulling N-rods and 3-in. tool from Hole No. 3 and setting casing.
	0950 - 1340	Drilling and setting casing to 210-ft penetration in Hole No. 3. HM left the site at 1000 hr.
	1340 - 1420	Installing closed-end 3-in. tool into Hole No. 3.
	1420 -1427	Driving closed-end 3-in. tool to test penetration.
	1427 - 1445	Setting up loading system.
	1445 - 1500	Performing immediate test at 210-ft penetration in Hole No. 3.
	1500 - 1515	Moving ram to Hole No. 4.
	1515 - 1620	Advancing casing to 230 ft in Hole No. 1. Drill crew left the site at 1620 hr.
	1620 - 2025	Performing X-probe 24-hr test at 120-ft penetration in Hole No. 4. Left the site at 2025 hr (JDB and WCVP).
April 22	0705 - 0740	Performing pull-out test at 120-ft penetration in Hole No. 4. Drill crew arrived at the site at 0715 hr.

Date	From To	Description of Activities
April 22	0740 - 0830	Pulling N-rods and X-probe from Hole No. 4.
	0830 - 0930	Drilling and installing casing to 141 ft in Hole No. 4.
	0930 - 1030	Installing X-probe to 141-ft test penetration in Hole No. 4. Drill crew left the site at 1030 hr.
	1030 - 1050	Performing immediate test at 141-ft penetration in Hole No. 4.
	1050 - 1400	Monitoring consolidation.
	1400 - 1530	Performing X-probe 4-hr test at 141-ft penetration in Hole No. 4.
	1530 - 1800	Monitoring consolidation.
April 23	0700 - 0750	Performing pull-out test on X-probe at 141 ft in Hole No. 4. Drill crew arrived at the site at 0720 hr.
	0750 - 0840	Pulling N-rods and X-probe from Hole No. 4.
	0840 - 1030	Drilling and setting casing to 160 ft in Hole No. 4.
	1030 - 1045	Preparing 3-in. tool (open-end).
	1045 - 1130	Installing open-end 3-in. tool to 160 ft in Hole No. 4. Driving occurred at 1120-1130 hr.
	1130 - 1150	Performing immediate test at 160-ft penetration in Hole No. 4.
	1150 - 1430	Drilling and setting casing to 230-ft test penetration in Hole No. 1.
	1430 - 1530	Installing X-probe into Hole No. 1. Drill crew left the site at 1530 hr.
	1530 - 1600	Performing immediate test at 230-ft penetration in Hole No. 1.
	1600 - 1830	Staying in the motel.
	1830 - 2000	Performing X-probe 4-hr test at 230-ft penetration in Hole No. 1. Left the site at 2000 hr.
April 24	0700 - 0740	Performing X-probe pull-out test at 230-ft penetration in Hole No. 1.
	0740 - 0800	Moving ram to Hole No. 3 (to perform 72-hr test at 210-ft penetration later on).

Date	From	To	Description of Activities
April 24	0800 -	0900	Pulling N-rods and X-probe from Hole No. 1. Started performing close-end 3-in. tool 72-hr test in Hole No. 3 at 0830 hr.
	0900 -	1030	Performing close-end 3-in. tool 72-hr test in Hole No. 3.
	1030 -	1040	Moving ram.
	1040 -	1120	Pulling N-rods and 3-in. tool from Hole No. 3.
	1120 -	1450	Drilling and setting casing to 230-ft test penetration in Hole No. 3.
	1450 -	1515	Preparing X-probe for the test.
	1515 -	1625	Installing X-probe into Hole No. 3.
	1625 -	1645	Performing immediate test at 230-ft penetration in Hole No. 3.
	1645 -	1730	Drilling and setting casing down to 250 ft in Hole No. 1. Drill crew left the site at 1730 hr.
	1730 -	1930	Staying at the motel.
	1930 -	2130	Performing X-probe 4-hr test at 230-ft penetration in Hole No. 3. During 2-way cyclic test, N-rods buckled due to lack of lateral support. Stop the testing at 2120 hr.
April 25	0700 -	0750	Performing X-probe pull-out test in Hole No. 3 at 230-ft penetration. Drill crew arrived at 0700 hr.
	0750 -	0850	Pulling N-rods and X-probe from Hole No. 3.
	0850 -	1030	Drilling and setting casing down to 250-ft penetration in Hole No. 1.
	1030 -	1120	Installing X-probe into Hole No. 1 to 250-ft test penetration.
	1120 -	1140	Performing immediate test.
	1140 -	1500	Pulling 4-in. casing from Hole No. 3. Drill crew left the site at 1500 hr.
	1500 -	1800	Monitoring pore pressure dissipation. Planning for the demobilization to Houston.

Date	From To	Description of Activities
April 26	0700 - 0740	Moving ram from Hole No. 1 to Hole No. 4. Drill crew arrived at the site at 0700 hr.
	0740 - 1130	Performing open-end 3-in. tool 72-hr test at 160-ft penetration in Hole No. 4.
	1130- 1230	Pulling N-rods and 3-in. tool from Hole No. 4.
	1230 - 1600	Performing X-probe 24-hr test at 250-ft penetration in Hole No. 1. Meantime, pulling 4-in. casing from Hole No. 4.
	1600 - 1700	Pulling N-rods and X-probe from Hole No. 1. LP&L cut our electricity at 1630 hr.
	1700 - 1900	Pulling 4-in. casing from Hole No. 1. Pulling ground anchors and grouting 2 holes (0-25 ft of Hole No. 1 and 3). Packing up everything.
April 27	0700 - 1000	Grouting 2 holes (0-25 ft in Hole No. 2 and 4). Demobilizing for Capozzoli drill crew. Drill crew left the site for another job at 1000 hr.
	0800 - 1030	Trucks arrived at 0905 hr. Crane arrived at 0915 hr. Demobilizing Earth Technology equipment. Crane left the site at 1020 hr. Trucks left the site for Houston at 1030 hr.
	1030 - 1130	Cleaning the site. JDB and FB left for MSY at 1130 hr.
	1130 - 1150	Checking out from motel (WCVP).
	1150 - 1600	Traveling to Baton Rouge, La.
	1600 - 1700	Visiting Capozzoli's office in Baton Rouge, La (WCVP).
	1700 - 2400	Traveling to Houston, TX.
April 28	0000 - 0130	Traveling to Houston. Arrived in Houston safely at 0130 hr.
April 29	- -	
April 30	0930 - 1500	Demobilizing in Houston. Trucks arrived at 1030 hr. Stored everything inside the warehouse. Returned the van at 1330 hr. The field operation phase of the Empire project was successfully completed.

APPENDIX B: X-PROBE EXPERIMENT AT THE 120-FT DEPTH

RESULTS OF THE X-PROBE EXPERIMENT AT THE 120-FT DEPTH

The experiment with an X-probe at the 120-ft depth was performed between the times of 1700 on 20 April and 0800 on 22 April 1984.

Installation of the probe was accomplished by using the draw-down on the drilling rig. The installation of the probe was completed at 1741 hours on 20 April. The maximum total pressure which was measured after the insertion of the probe was 20.4 ksf; the corresponding maximum pore pressure was 18.3 ksf.

The variation in soil pressures during consolidation are given in Plate B-1. As shown in the plate, consolidation was interrupted by load tests at three different times. The first load test was begun at 1755, 14 minutes after installation. The major series of load tests were begun at 1629 on 21 April, 22 hours and 48 minutes after installation. Approximately 3 1/2 hours were required to complete the series of static and one-way cyclic load tests. The probe was then allowed to remain in place from 2000 on 21 April until 0738 on 22 April, at which time the probe was loaded to failure in tension and compression, then removed from the boring.

As shown in Plate B-1, both the total pressure and the pore pressure began to decrease immediately after insertion. At the time the immediate test was performed, the total pressure had decreased to 19.2 ksf, with an accompanying pore pressure of 16.8 ksf, yielding a value of radial effective pressure of 2.4 ksf.

It should be noted that, since the data are plotted against the logarithm of time, the first point on the curve is the value recorded one minute after installation; because of the rapid decrease in the pressures during this time, the values on the figures do not always correspond with the values of maximum pressure which are given in the text and tabulated in Volume I.

The value shown in the plate as the assumed value of ambient pore pressure is 6.5 ksf, which is 0.87 ksf greater than that which would be calculated for a hydrostatic condition using an assumed unit weight of water of 63.5 lb/cu ft. Based on the value of ambient pore pressure shown in the plate, the maximum excess pore pressure created during insertion was approximately 9.8 ksf, or approximately 10 times the undrained shear strength at this depth.

The load test performed 14 minutes after insertion consisted of a loading to failure in tension, one load reversal to failure in compression, then a second reversal, with the upward displacement continued until the slip-joint at the probe tip was in the mid-range of travel. This procedure was followed for each installation of a probe, so that the two-way cyclic tests could be performed near the same initial position as the load test after consolidation.

The results of the test are shown in Plate B-2, which is a reproduction of the analog record. The peak shear transfer on the first loading was 0.42 ksf, with a residual shear transfer equal to 0.36 ksf being recorded after the second reversal.

As seen in Plate B-1, the single cycle of reversed shearing resulted in an increase in the pore pressure and an accompanying reduction in the effective pressure. The values of radial soil pressures at the end of the immediate test were: a total pressure of 19.0 ksf and a pore pressure of 16.9 ksf, yielding a value of 2.1 ksf for the effective pressure.

The soil was then allowed to consolidate for almost 23 hours. At the end of this period, the radial soil pressures showed values of 15.6 ksf for the total pressure and 10.1 ksf for the pore pressure, yielding a value of radial effective pressure of 5.5 ksf. Based on the value of ambient pore pressure shown in Plate B-1, the percentage of dissipation of the excess pore pressure was 84 percent, near the targeted value of 90 percent.

The results of the load test after consolidation are given in Plate B-3. The maximum shear transfer during the initial laoding was 0.96 ksf, which is approximately equal to the undrained shear strength at this depth as given in the interpreted shear strength profile in Volume I.

Following the tension test to failure, the probe was unloaded to near-zero shear. The pore pressure was then monitored until it had returned to a value near that which had been observed prior to loading. At the end of the test to failure and the subsequent unloading, the soil pressures were: a total radial pressure of 15.4 ksf, a pore pressure of 11.1 ksf, and a value of radial effective pressure of 4.3 ksf. The test to failure thus resulted in an increase in the pore pressure of 1.0 ksf, a reduction in total radial pressure of 0.2 ksf, yielding a reduction in the radial effective pressure of 1.2 ksf.

The elevated pore pressure rapidly dissipated, reducing to 10.5 ksf after a period of 19 minutes. One hour and fourteen minutes after the test, the pore pressure was 10.3 ksf. At this time, it was felt that the value was close enough to the value of 10.1 ksf measured prior to the disturbance so that the subsequent tests would not be affected by the excess pore pressures. Therefore, the one-way cyclic tests were begun.

At the beginning of the one-way cyclic tests, the total radial pressure was 15.3 ksf with an accompanying pore pressure of 10.3 ksf, yielding a radial effective pressure of 5.0 ksf.

A summary of the one-way cyclic tension tests is given in Plate B-4. The lower figure contains the values of maximum and minimum shear transfer on each cycle. The displacements corresponding to the peak values of shear are shown in the upper figure. As shown in the plate, the application of repeated tension loads with values exceeding 60 percent of the maximum value recorded during the first test resulted in the accumulation of progressive upward displacement. The rate of accumulation of permanent displacement increased when the peak shear was increased to a value equal to 96 percent of the maximum value on the first loading to failure.

The progressive pullout of the probe during the cyclic tension tests is due to two factors: the short length of the probe and the nonlinear, inelastic nature of the t-z response. Because of the inelastic nature of the shear-displacement response, even at small displacement, a small amount of plastic deformation is accumulated on each loading. Since the probe is fairly short, with no elastic member below the probe to restrain the movement, the probe is progressively pulled upward.

The linear rate of accumulation of permanent displacement indicates that no degradation in resistance accompanied the progressive pullout; had any losses in resistance occurred, the rate of accumulation of displacement would have increased with the increasing number of cycles. The load test to failure after this sequence of repeated loadings supports this conclusion; no loss in shear resistance was evident.

The behavior pattern shown in Plate B-4 would not occur if a similar pattern of loading were applied to the top of a long, elastic pile. Even though the same degree of inelasticity in the t-z response were to be exhibited in the soil adjacent to a long pile, the pile itself would not accumulate any progressive upward displacement. The effect on the pile would be merely a slight redistribution of axial stress along the embedded length. Because of the decreases in the magnitude of axial stress in the pile with depth, the peak-to-peak pile displacements during the repeated loading will also decrease; the soil along the lower portions of the pile will thus behave elastically, and will, in effect, anchor the upper portion of the pile, preventing any progressive upward movement.

Such would not be the case if the pile were short and were sufficiently stiff, so that inelastic soil response would be produced along a significant portion of its length. For such a pile, the behavior would be exactly as shown for the X-probe: a progressive upward movement during each application of the peak tension load.

The fluctuations in the radial soil pressures during the one-way cyclic tests are shown in Plate B-5. The figure also contains the changes in the soil pressures during the initial loading to failure. As seen in the figure, the repeated cyclic tension loading caused fluctuations in the radial pressure. The cumulative effects on the radial pressures were: a reduction in the radial total pressure of 0.2 ksf, an increase in the pore pressure of 0.2 ksf, leading to a decrease in the radial effective pressure of 0.4 ksf, less than the decrease of 1.2 ksf during the initial load test to failure.

Upon completion of the one-way tension tests, the probe was loaded to failure in tension. The results of the test are shown in Plate B-6. As shown in the figure, the maximum resistance recorded after the one-way cyclic tests was the same as that during the initial loading, i.e., 0.97 ksf. Thus, the application of the many cycles of loading, in which 0.040 in. of plastic deformation was accumulated, had no effect on the maximum shear transfer.

The probe was then subjected to five cycles of loading to failure in tension and compression, with the results shown in Plate B-7. The initial maximum shear transfer on the first cycle was 0.95 ksf; the peak value on the fifth cycle had reduced to 0.91 ksf. The residual shear transfer during plastic slip was 0.80 ksf.

After five cycles of loading, the displacement limits were increased from $\pm$0.05 in. to $\pm$0.1 in., and the rate of loading was increased. The displacement limits were increased in order to enable the digital system to obtain values of peak resistance during cycling. Each cycle of loading lasted only 24 seconds. With a minimum sample time of 1 second, only 24 samples were obtained during each cycle. Because of the accumulation of elastic deformation in the N-rod string, most of the digital data points were obtained during the period of load reversal; the period of plastic slip lasted only 4 seconds, yielding only 3 or 4 samples during the plastic portion of each half-cycle.

The results of the fast-rate loading are shown in Plate B-8. As can be seen in the figure, seven cycles of load reversal were applied. Cyclic degradation in resistance continued during the fast-rate cycling, with the value of shear transfer at yield on the last cycle being 0.88 ksf, and a minimum value of residual shear of 0.79 ksf.

The testing was then stopped, the displacement limits reduced to approximately $\pm$0.05 in. and two cycles of reversed-shear loading were applied. The results of the two cycles of loading are shown in Plate B-9. The peak shear transfer on the second cycle was 0.88 ksf; the residual value was 0.72 ksf.

During the fast-rate test, the rate of slip was 0.032 in./sec; during the slow cycles, the rate was 0.0020 in./sec. The increase in rate was thus 1.2 log cycles, with an increase in the plastic shear of 0.07 ksf, yielding an increase of 8 percent per log cycle of increase in slip rate.

A comparison of Plates B-8 and B-9 shows that in this experiment the clay acted as a Bingham solid, in that one peak values of shear transfer at yield were not affected by the rate of loading, being 0.88 ksf during both the fast-rate and slow-rate cycles; the effects of load rate were confined to the plastic-shear portion of the curves.

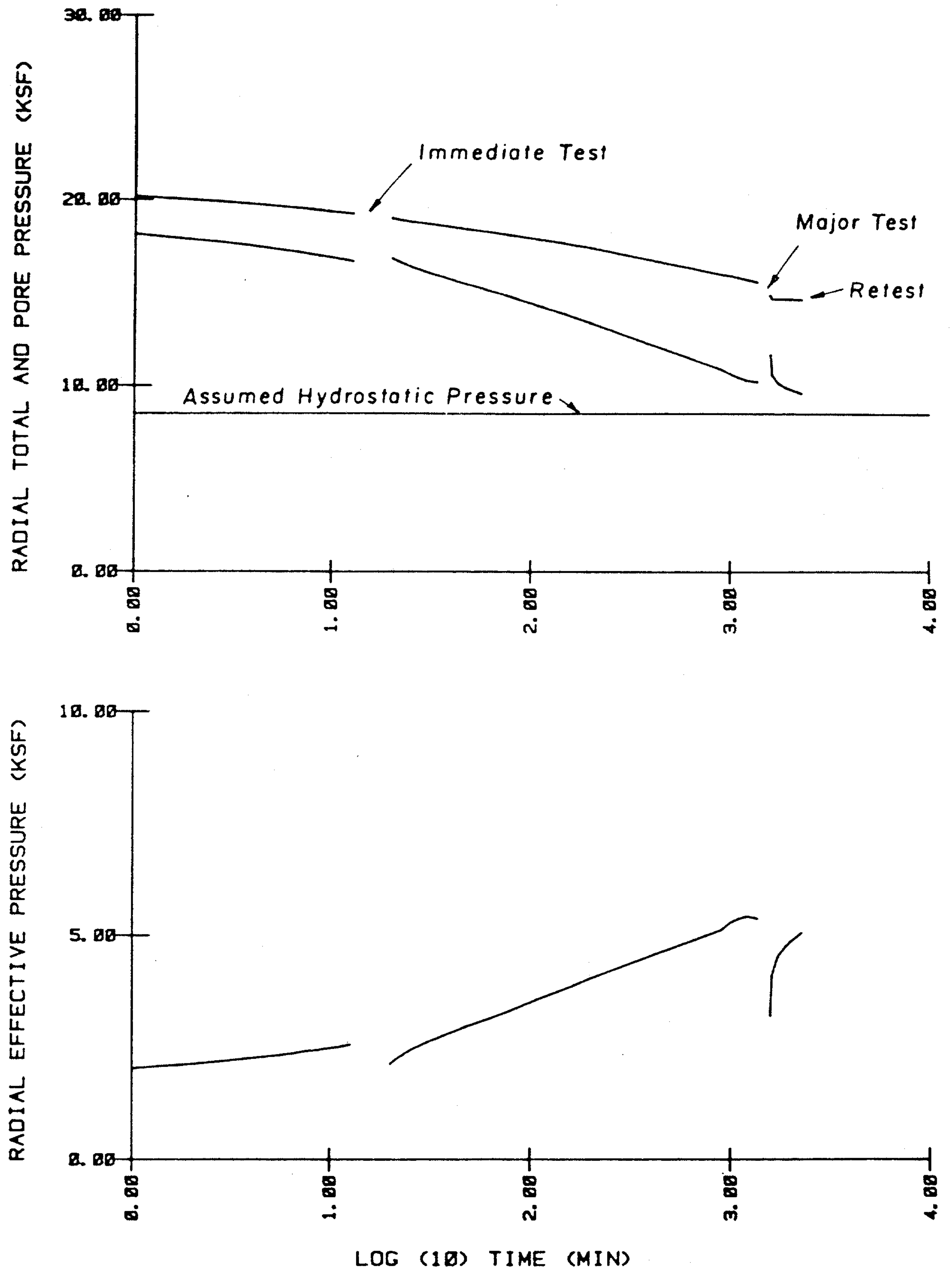
The fluctuations in the radial soil pressures during the two-way cyclic tests are shown in Plate B-10. As seen in the figure, the radial total pressure changes little during the cyclic shearing; the abrupt decreases in the pore pressure and the corresponding increases in radial effective pressure occur during plastic slip, as was shown in Volume I. During the cyclic tests at the faster load rate, the fluctuations in the pressures increase in magnitude. This pattern of behavior was consistent throughout all the fast-rate tests.

Upon completion of the major series of tests, the probe was left undisturbed for a period of 11 1/2 hours. As shown in Plate B-1, the pore pessures which were generated during the cyclic tests quickly dissipated, reducing from a value of 12.0 ksf at the end of the cyclic tests to 9.6 ksf. As shown in the plate, an increase in radial effective pressure accompanied the dissipation of the excess pore pressures, with a value of 5.1 ksf being recorded prior to the load test. At this time, the dissipation of the excess pore pressure was about 89 percent complete.

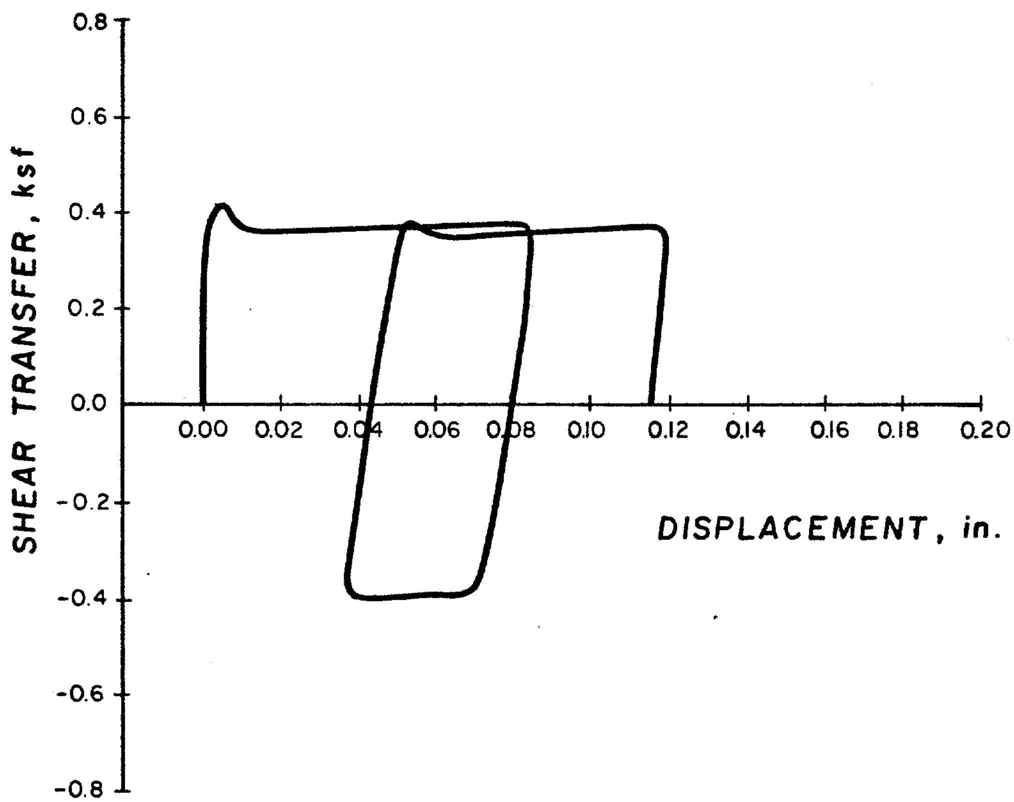
It should be noted in Plate B-1 that the pore pressure dissipation curve returns to that which was established prior to the cyclic tests, suggesting that the cyclic loading did not greatly affect the soil surrounding the pile. Had a considerable portion of clay been affected, the consolidation behavior should not have returned to the previous path after the test. Instead, the cyclic loading seems to create only a temporary perturbation of the process, but not a severe disruption.

The results of the load test performed after the 11 1/2 hours of additional consolidation are shown in Plate B-11. The peak resistance on the first loading was 1.26 ksf, with a residual shear on the first loading of 0.92 ksf; the residual shear during the second loading was 0.89 ksf.

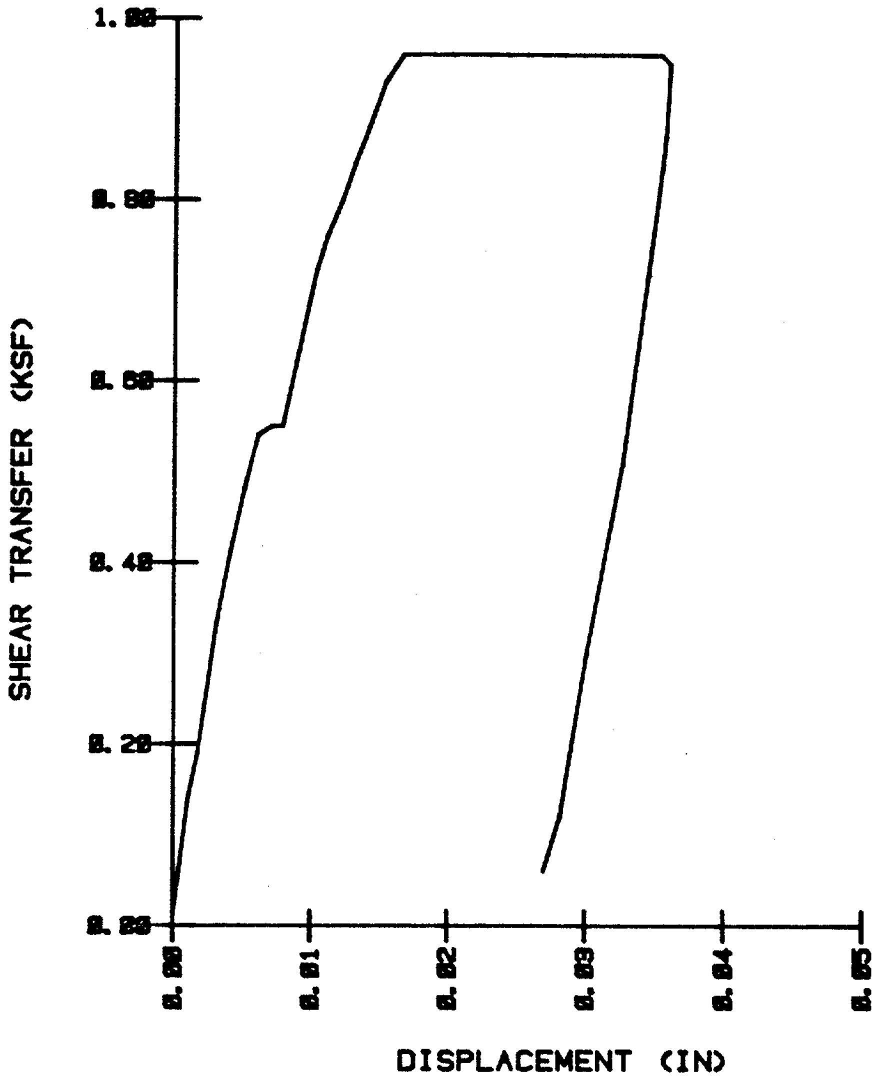
Upon completion of this test, the probe was removed. Although not shown in Plate B-11, the upward displacement was continued until the probe was free of the soil.



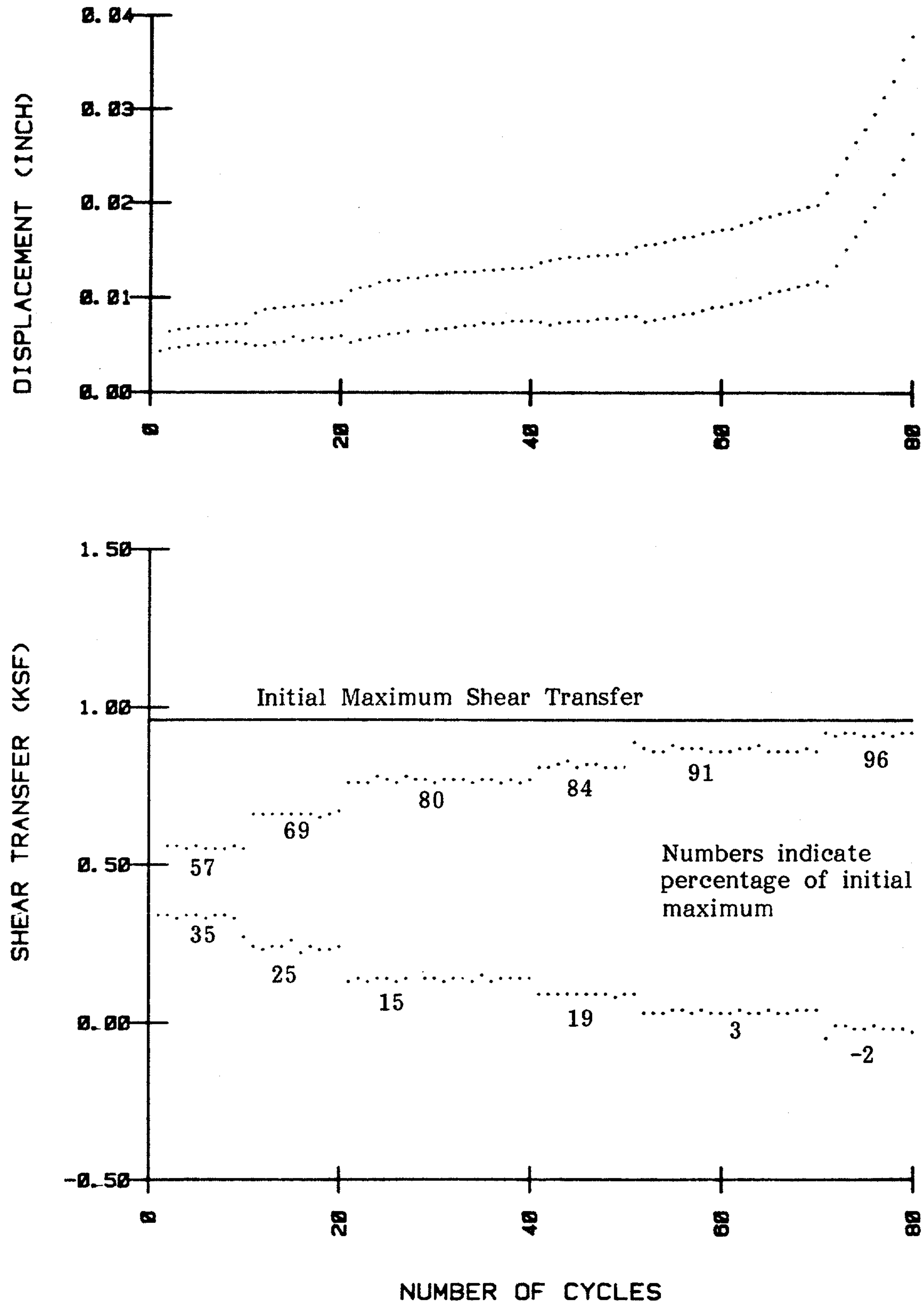
VARIATIONS OF SOIL PRESSURES DURING CONSOLIDATION



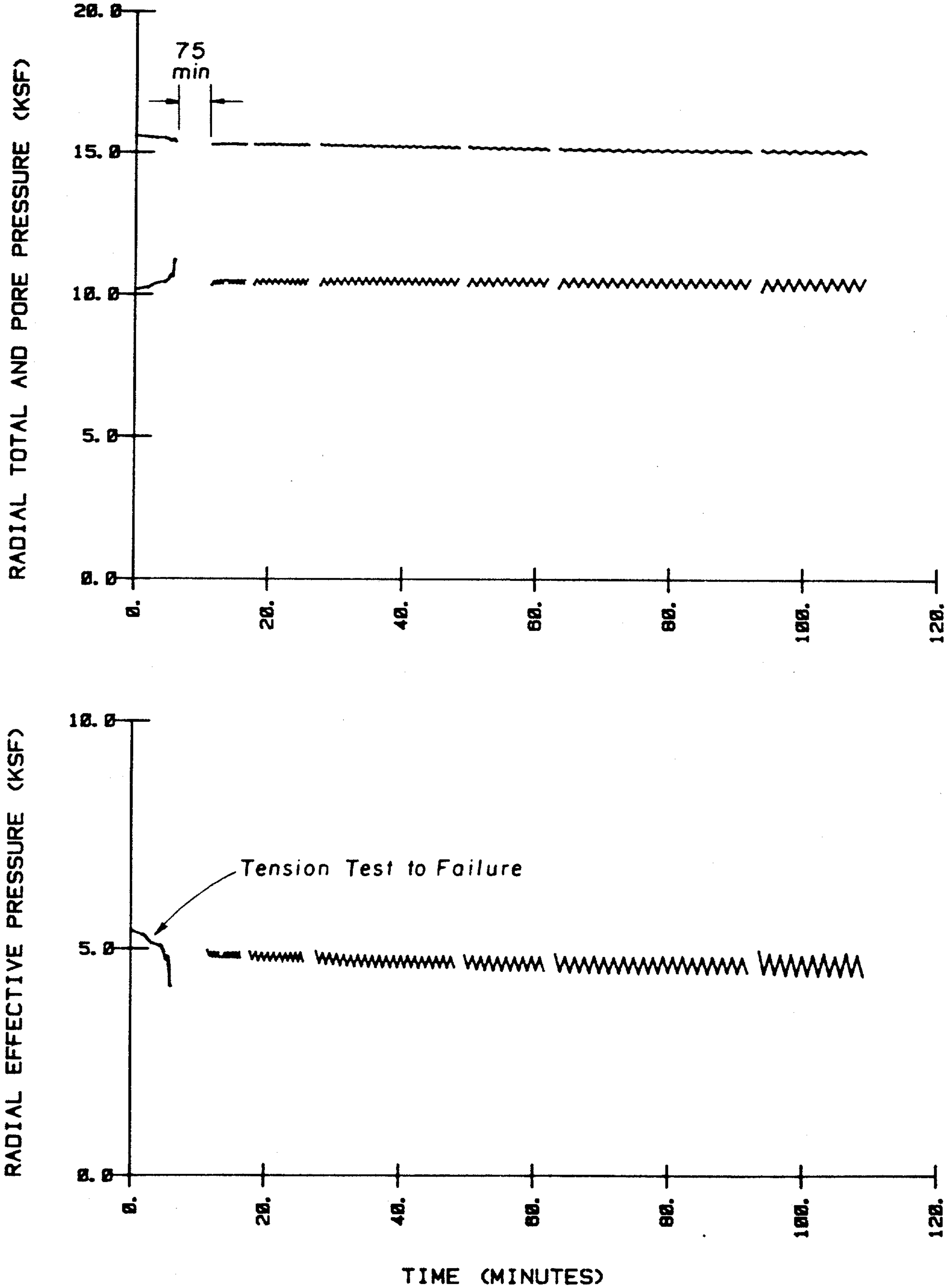
RESULTS OF THE IMMEDIATE LOAD TEST



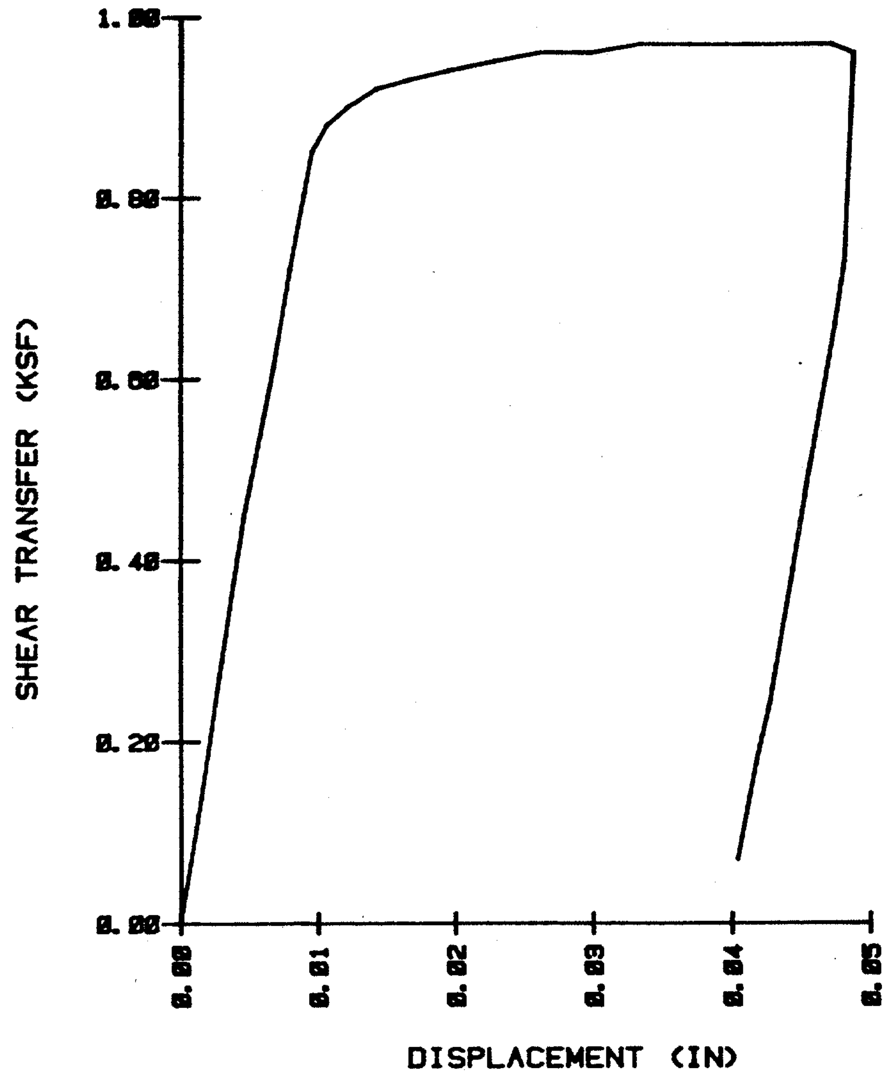
INITIAL LOADING TO FAILURE AFTER CONSOLIDATION



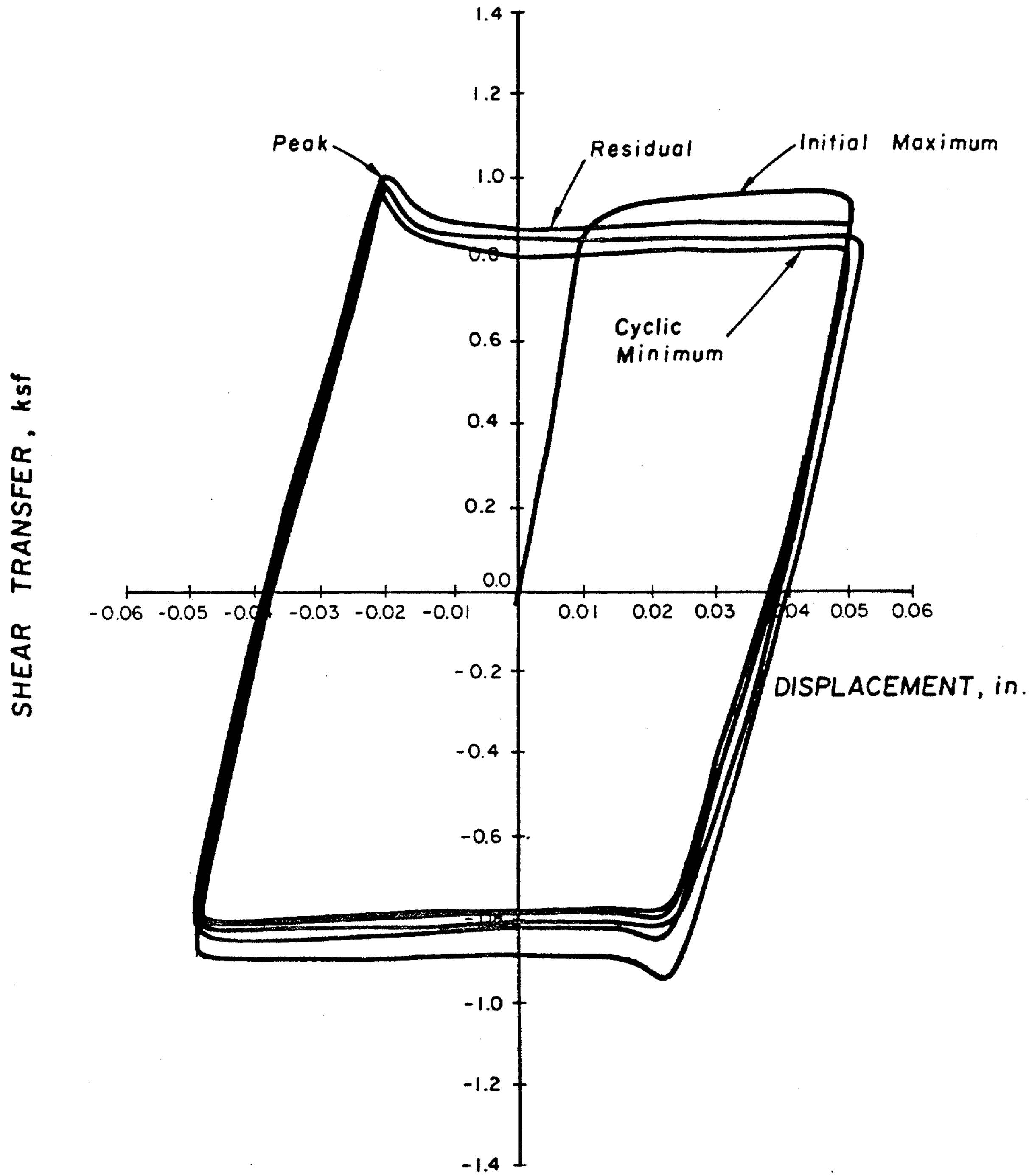
ONE-WAY CYCLIC TENSION TEST RESULTS



PRESSURE FLUCTUATIONS DURING ONE-WAY TENSION TESTS

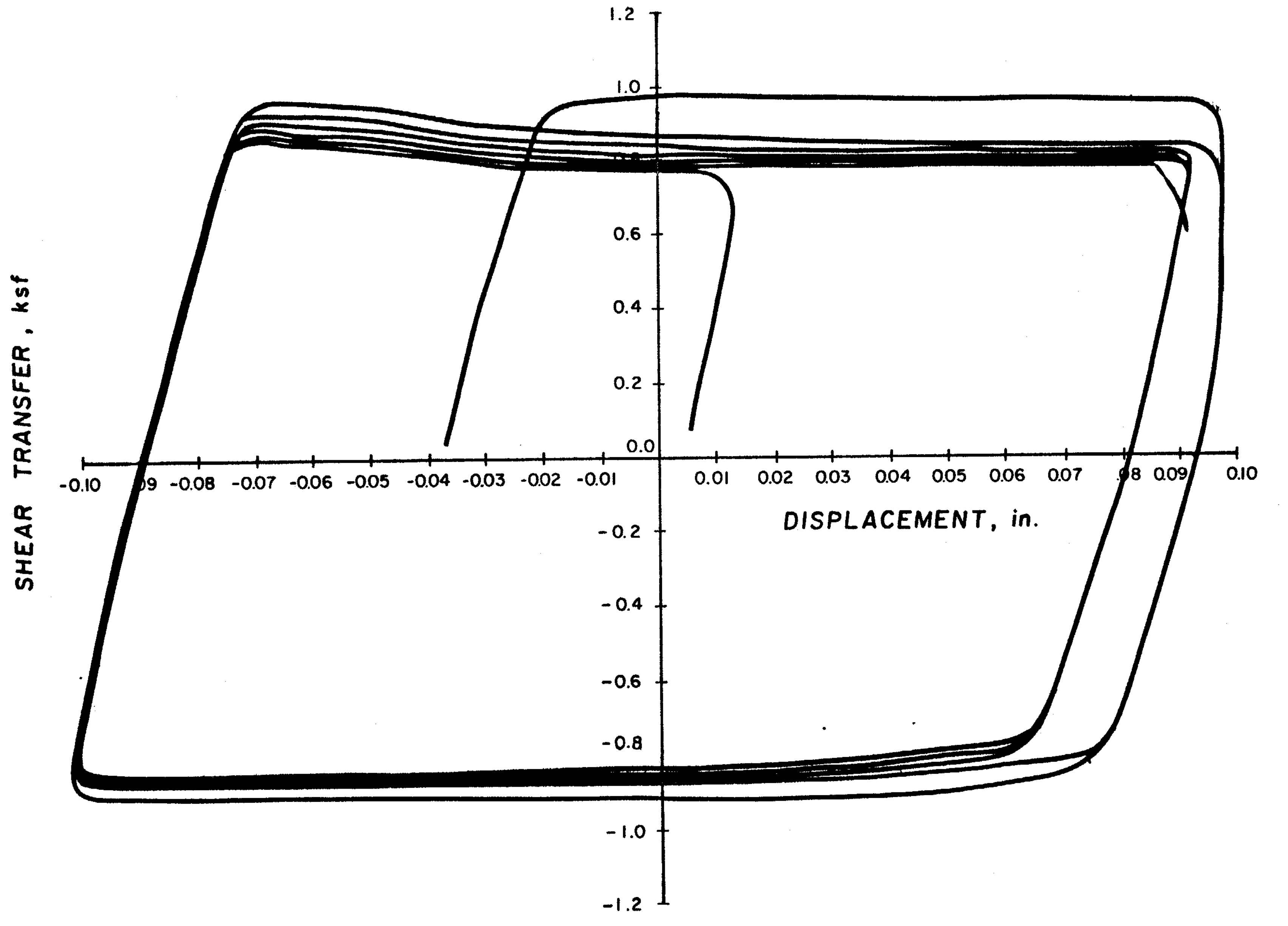


RESULTS OF THE TENSION TEST TO FAILURE
AFTER THE ONE-WAY CYCLIC TESTS

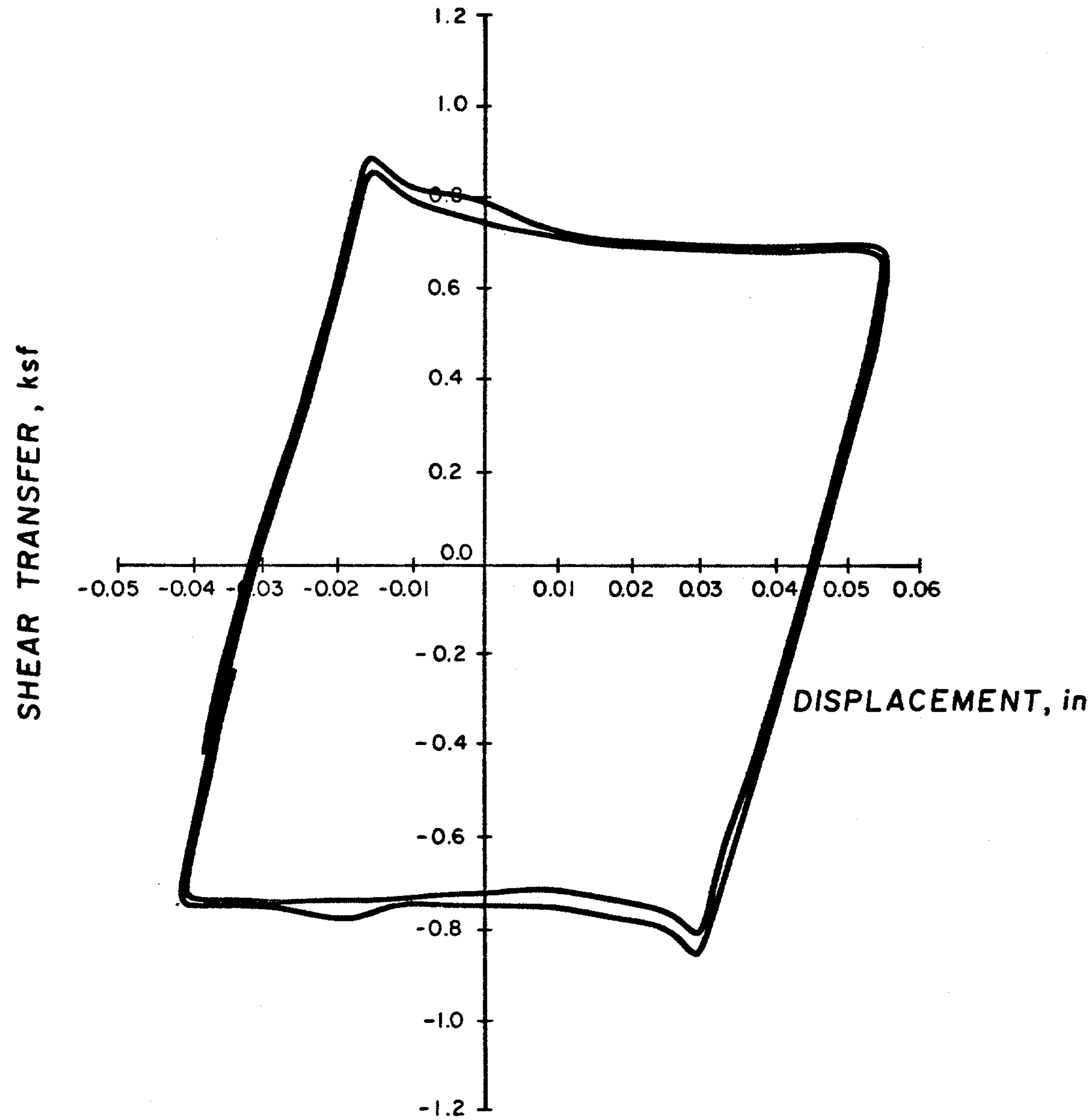


RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST

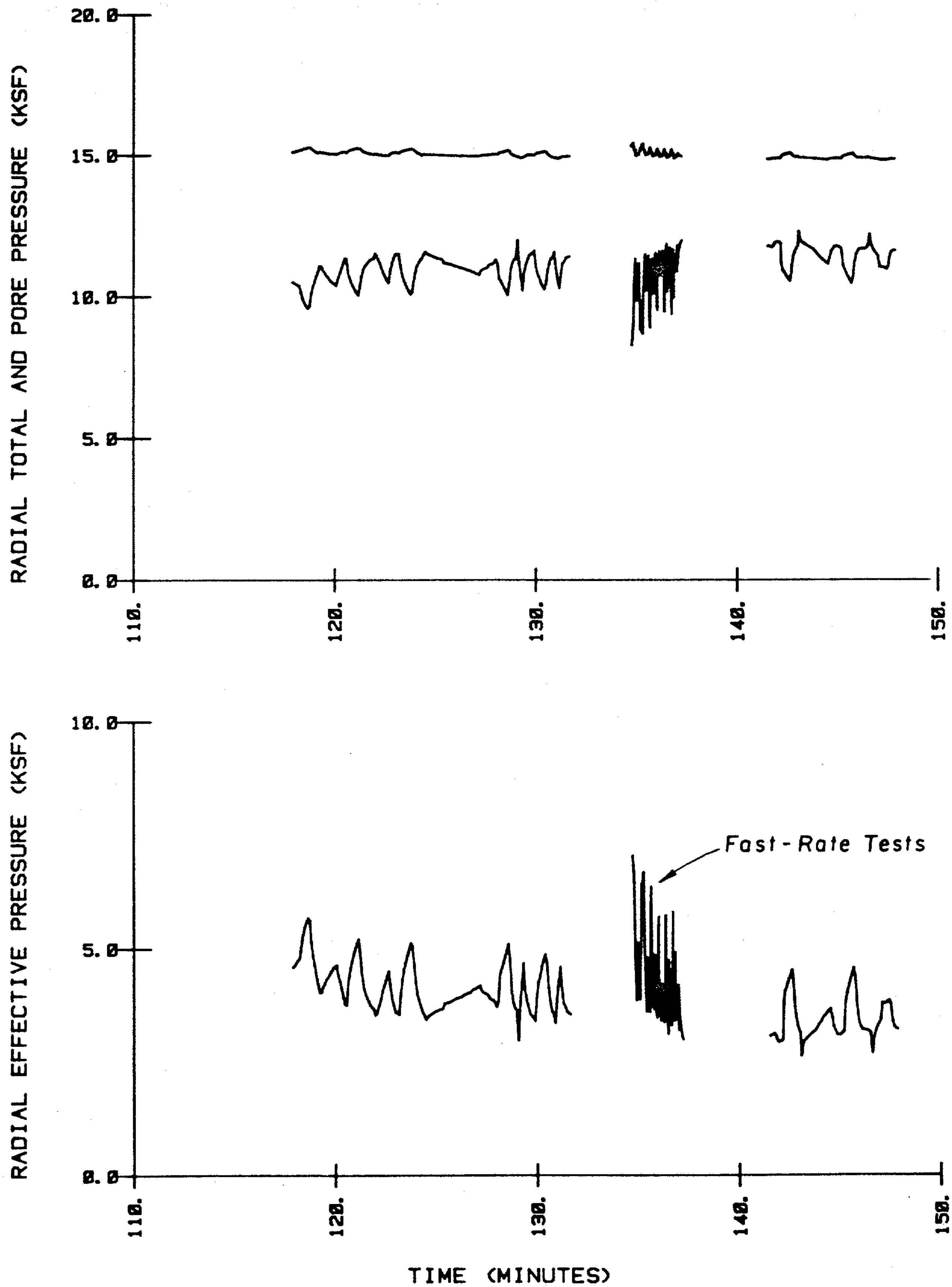
JOB NO.



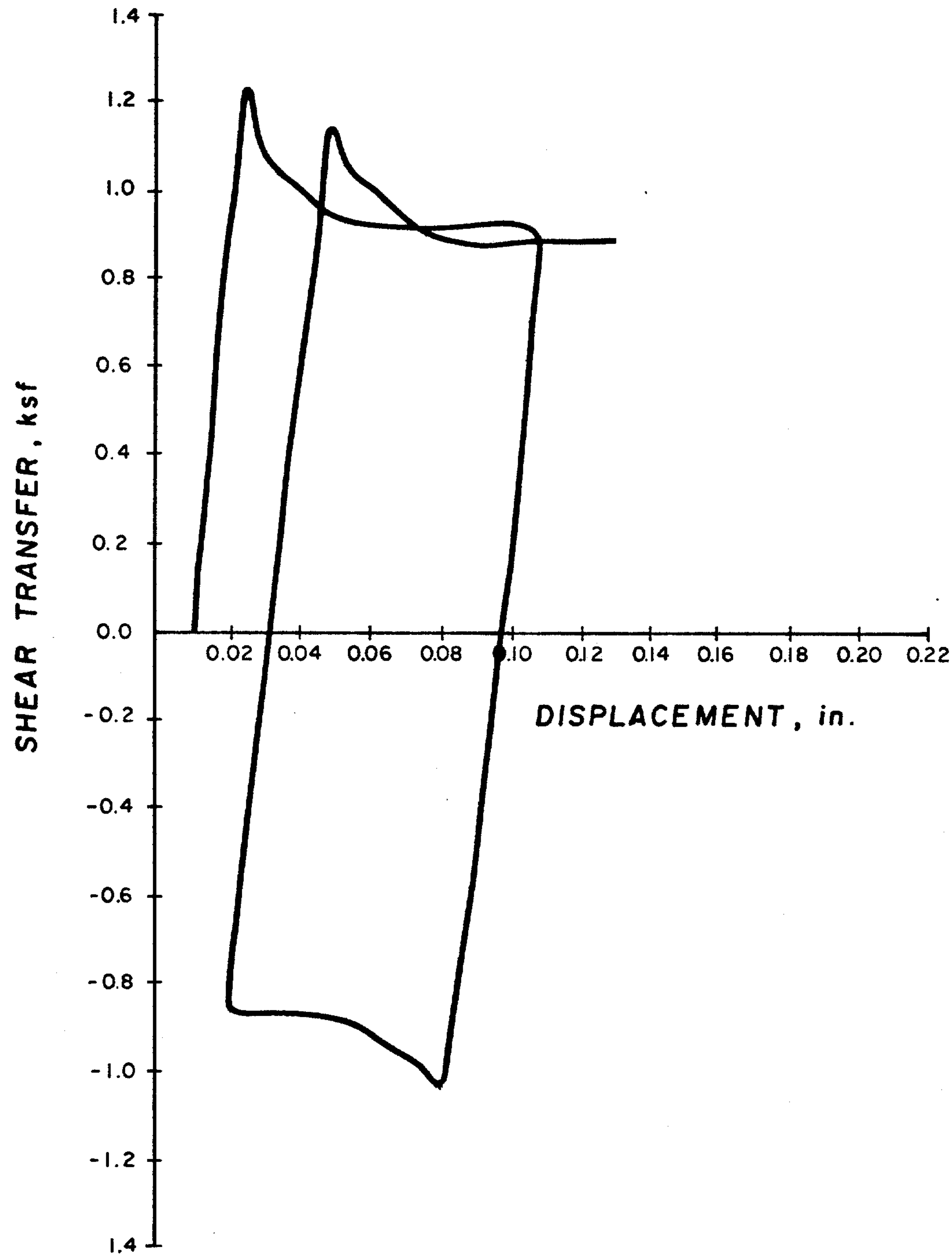
RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST



REPEAT OF SLOW-RATE TWO-WAY CYCLIC TEST



PRESSURE FLUCTUATIONS DURING TWO-WAY CYCLIC TESTS



TESTS TO FAILURE AFTER ADDITIONAL CONSOLIDATION

APPENDIX C: FIRST X-PROBE EXPERIMENT AT THE 141-FT DEPTH

RESULTS OF THE FIRST X-PROBE EXPERIMENT AT THE 141-FT DEPTH

The first experiment using an X-probe was performed during the period from 1500 hours on 16 April until 0800 hours on 17 April.

The probe was installed by pushing the top of the N-rod string with the draw-down on the drilling rig. The installation was completed at 1523 on 16 April.

The variation in soil pressures during consolidation are shown in Plate C-1. As shown in the plate, both the total radial pressure and the pore pressure decreased, with the radial effective pressure showing a reasonably-linear trend. The consolidation was twice interrupted by load tests, but the trend lines in the pressure-log time curves follow generally-consistent patterns, with the curves returning to the trends established prior to loading.

The first load test was begun 15 minutes after installation. At this time, the total pressure had decreased from a maximum value of 23.8 ksf to 22.3 ksf; the pore pressure had decreased from a maximum of 21.9 ksf to 19.5 ksf; the radial effective pressure thus increased from 1.9 ksf to 2.9 ksf at the time of the test.

The results of the test performed 15 minutes after installation are shown in Plate C-2. The peak shear on the first loading was 0.38 ksf; the residual shear after the single cycle of reversed loading was 0.35 ksf.

The effects of the test on the pressures can be seen in Plate C-1, and include: an increase in the pore pressure to 20.2 ksf; a reduction in the total pressure to 22.2 ksf; and a reduction in the radial effective pressure to 2.0 ksf.

The soil was allowed to consolidate until 1952 hours, at which time the total radial pressure had decreased to 20.2 ksf, the pore pressure had decreased to 15.6 ksf, with a corresponding increase in the radial effective pressure to 4.6 ksf. The percentage of dissipation of excess pore pressure was thus 52 percent, near the targeted value of 50 percent.

The probe was then subjected to a slow loading to failure in tension, with the results shown in Plate C-3. The maximum shear transfer was 0.76 ksf, which is 61 percent of the estimated undrained shear strength at this depth. The effects of the load test on the soil pressures were as follows: no change in the total pressure (20.2 ksf), an increase in the pore pressure, to 16.4 ksf and a decrease in the radial effective pressure, to 3.7 ksf.

Upon completion of this test, the pore pressures were monitored for a period of 34 minutes, at which time the total pressure was 19.9 ksf, the pore pressure was 15.5 ksf, yielding a radial effective pressure of 4.4 ksf.

The sequence of one-way cyclic (repeated) tension tests were then performed, with the results shown in Plate C-4. As shown in the plate, progressive accumulation of upward displacement began at load levels greater than 70 percent of the initial maximum, with the rate of accumulation becoming accelerated after the load level was increased to 113 percent of the initial maximum resistance.

During the last 3 applications of load, the minimum shear transfer increased to 46 percent of the initial maximum; this reduction in the cyclic component had no effect on the rate of accumulation of permanent displacement. The amount of permanent displacement which was accumulated on each cycle is a function of the magnitude and duration of the peak tension load only; the value of the minimum shear is not a factor.

The fluctuations in the pressures during the one-way tension tests are shown in Plate C-5. It can be noted in the plate that the total pressures and the pore pressures tended to decrease during the tests; the effective pressure increased somewhat. At the end of the one-way tension tests, the total radial pressure was 19.3 ksf; the pore pressure was 14.6 ksf, yielding a calculated radial effective pressure of 4.7 ksf.

The probe was then subjected to a slow loading to failure in tension, followed immediately by several cycles of two-way, controlled-displacement loading. The maximum resistance on the first cycle of loading was 0.91 ksf, an increase of 0.13 ksf (20 percent) in the maximum resistance. Using the value of 0.91 ksf as the yardstick by which to evaluate the results of the one-way repeated tension tests, the value of shear transfer during the last few cycles (0.86 ksf) was 95 percent of the static resistance at the time of loading, rather than the value of 113 percent shown which was based on the initial resistance.

The results of the two-way cyclic tests are shown in Plate C-6. The shapes of the curves during tension loading at values of displacement less than about -0.02 in. are a consequence of load rate effects, as discussed in Volume 1. As discussed in the preceding section, the value of peak resistance is not affected; only the residual shear, and only during the period of rapid upward travel.

The peak resistance shown for the fifth cycle of loading is 0.88 ksf; the residual shear on the same cycle is 0.71 ksf.

The rate of loading was then increased, with the results shown in Plate C-7. The peak resistance on the last cycle of loading was 0.84 ksf, with a value of residual shear of 0.75 ksf.

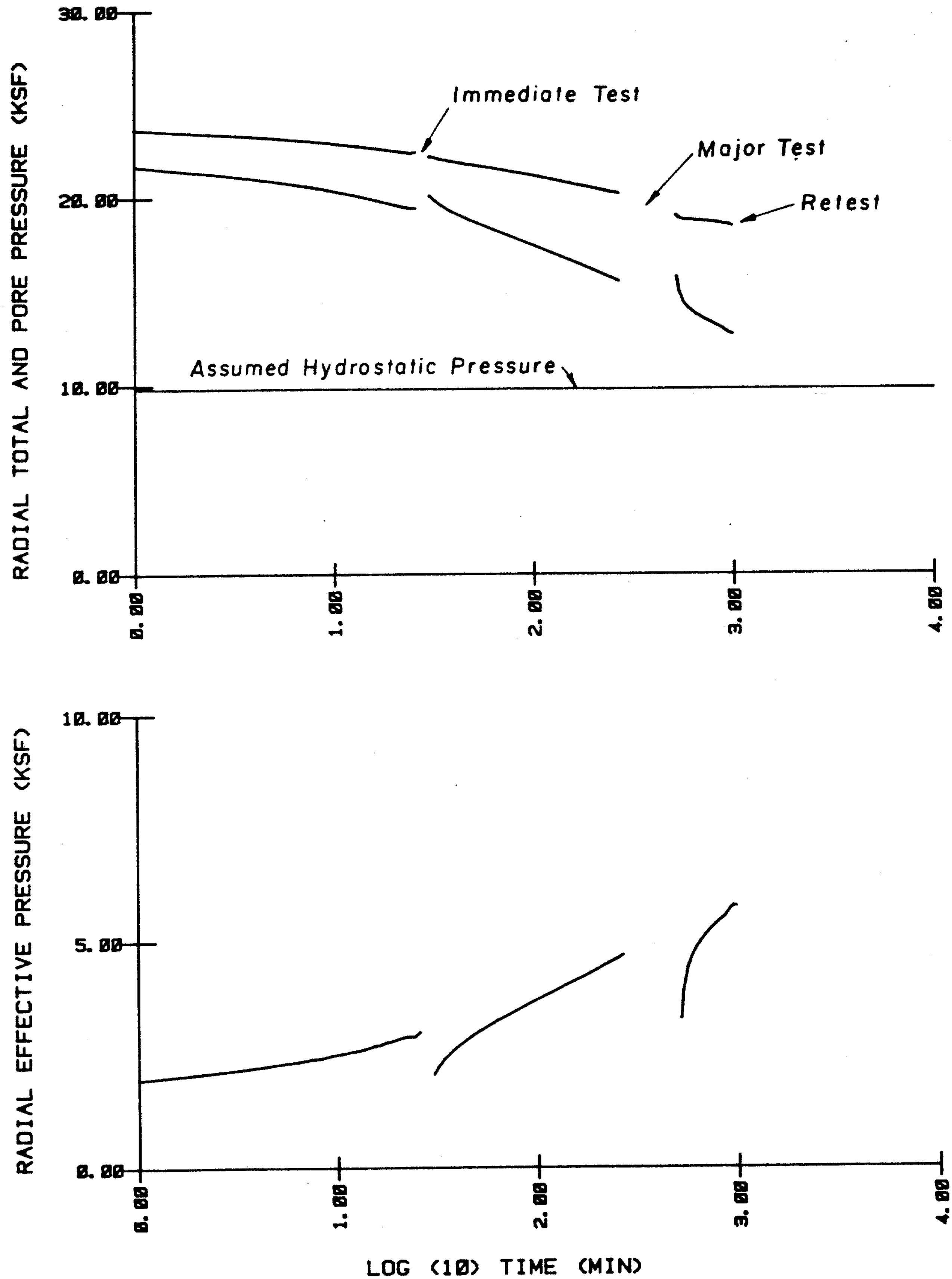
The rate of loading was then decreased, to the original slower rate, with the results shown in Plate C-8. The peak resistance shown is 0.86 ksf; the value of residual shear, 0.70 ksf.

The effects of load rate are, again, those defined for an ideal Bingham solid that is the yield stress is not affected by load rate; only the value of plastic shear after yield is.

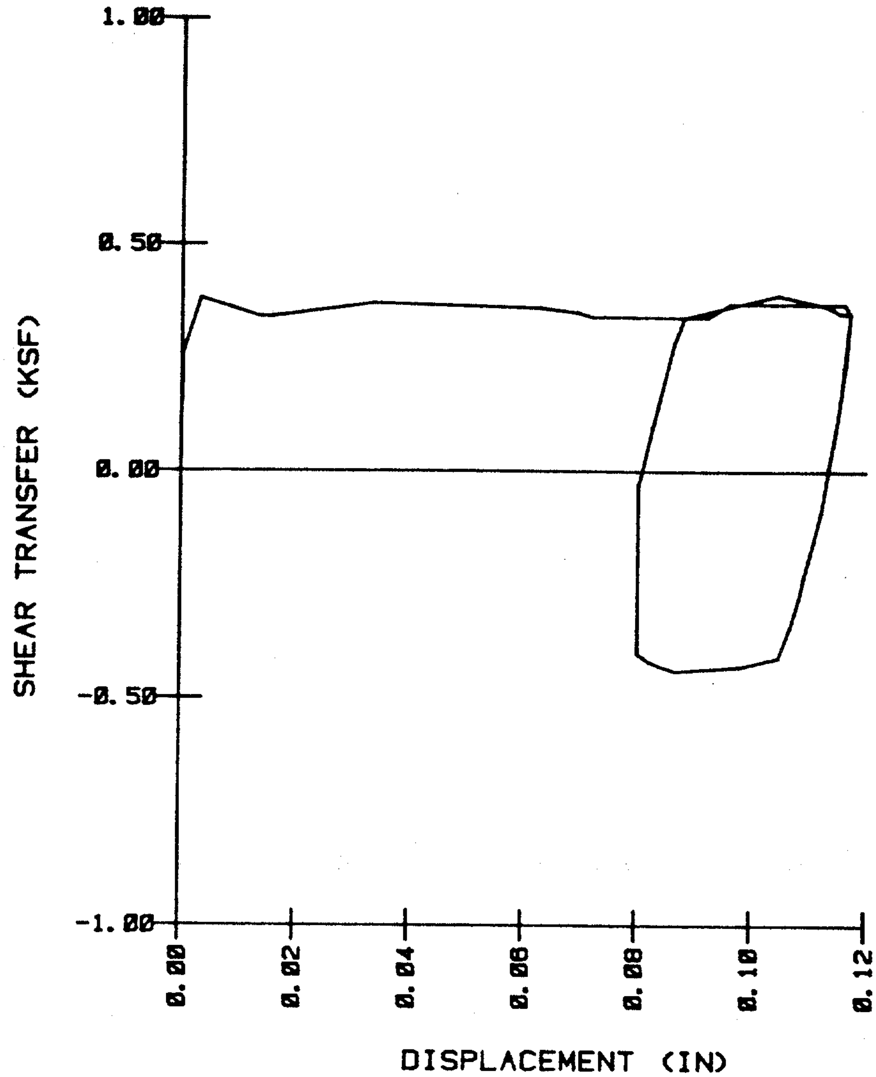
The fluctuations in the soil pressures during the two-way cyclic tests are shown inPlate C-9. Again, the radial total pressure is not much affected by the cyclic loading; the pore pressures show a sharp decrease during plastic shear, with an accompanying increase in the radial effective pressure.

A shown in Plate C-1, the soil was then allowed to continue to consolidate for a period of 8 hours, at which time the probe had to be removed in order to proceed with the experiments. At this time, the total pressure had decreased from a value of 19.1 ksf at the end of the cyclic tests to a value of 18.5 ksf. The pore pressure also decreased, from 15.6 ksf to 12.7 ksf, with a corresponding increase in the radial effective pressure from 3.5 ksf to a value of 5.8 ksf.

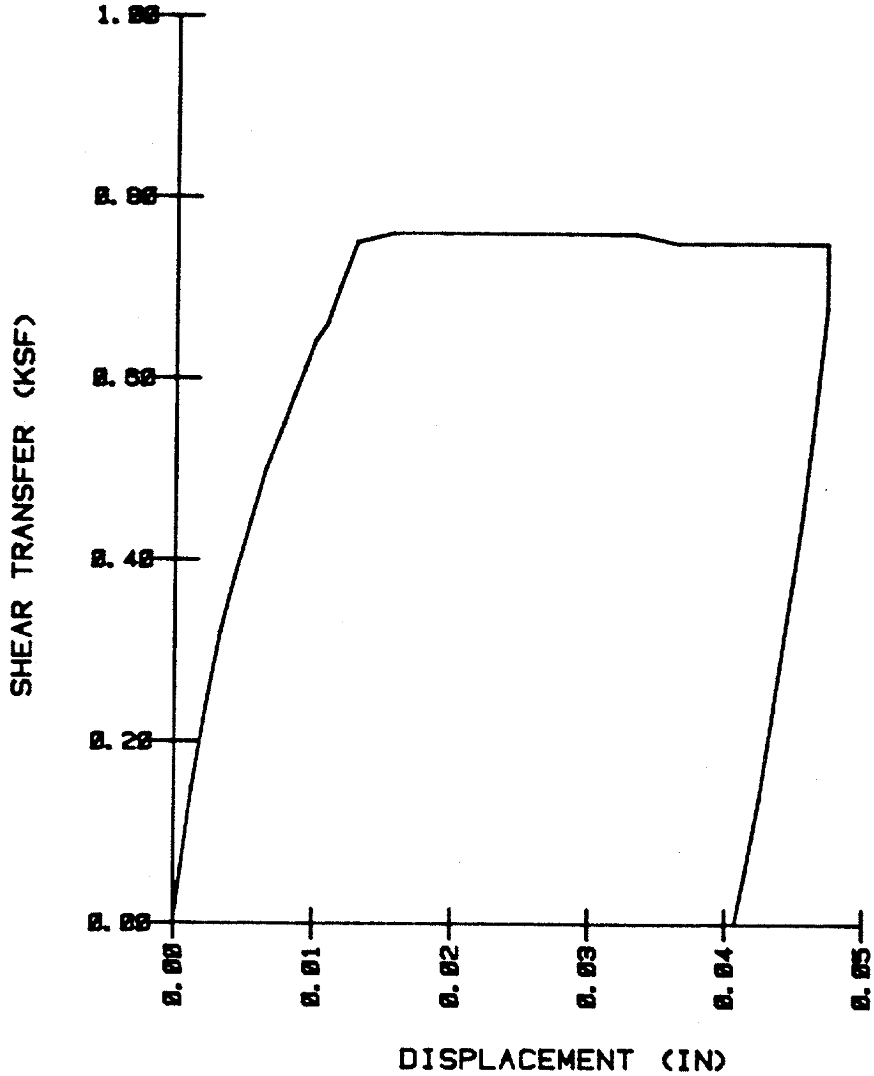
The probe was then subjected to a slow loading to failure in tension, followed by two cycles of reversed loading, then removed from the boring. The results of the test are shown in Plate C-10. The peak resistance on the first loading to failure was 1.28 ksf; very near the value of 1.25 ksf given for the undrained shear strength at this depth. The residual shear on the first loading was 0.96 ksf; this reduced to 0.82 ksf on the third cycle.



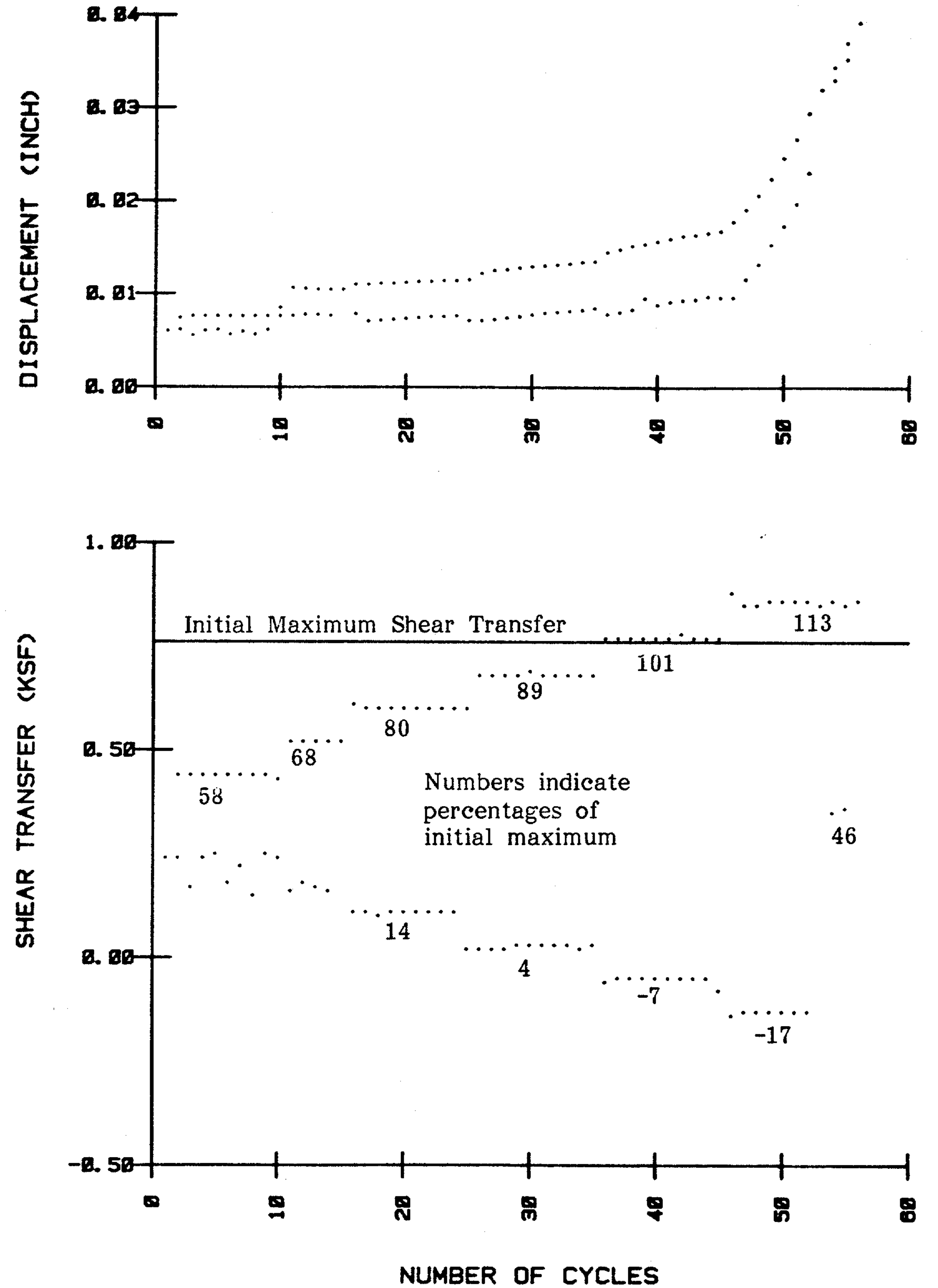
VARIATIONS IN THE SOIL PRESSURES DURING CONSOLIDATION



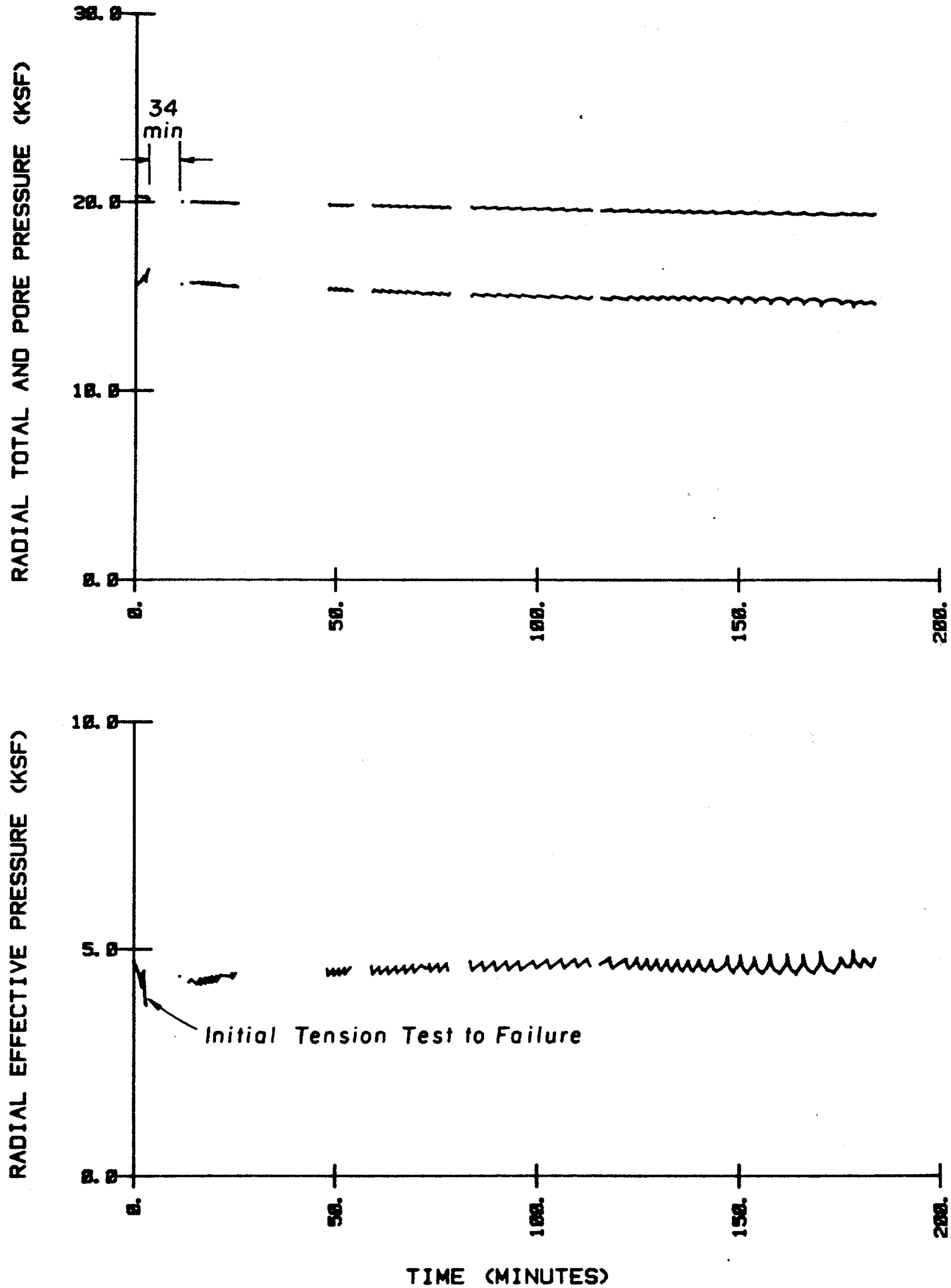
RESULTS OF THE LOAD TESTS IMMEDIATELY AFTER INSTALLATION



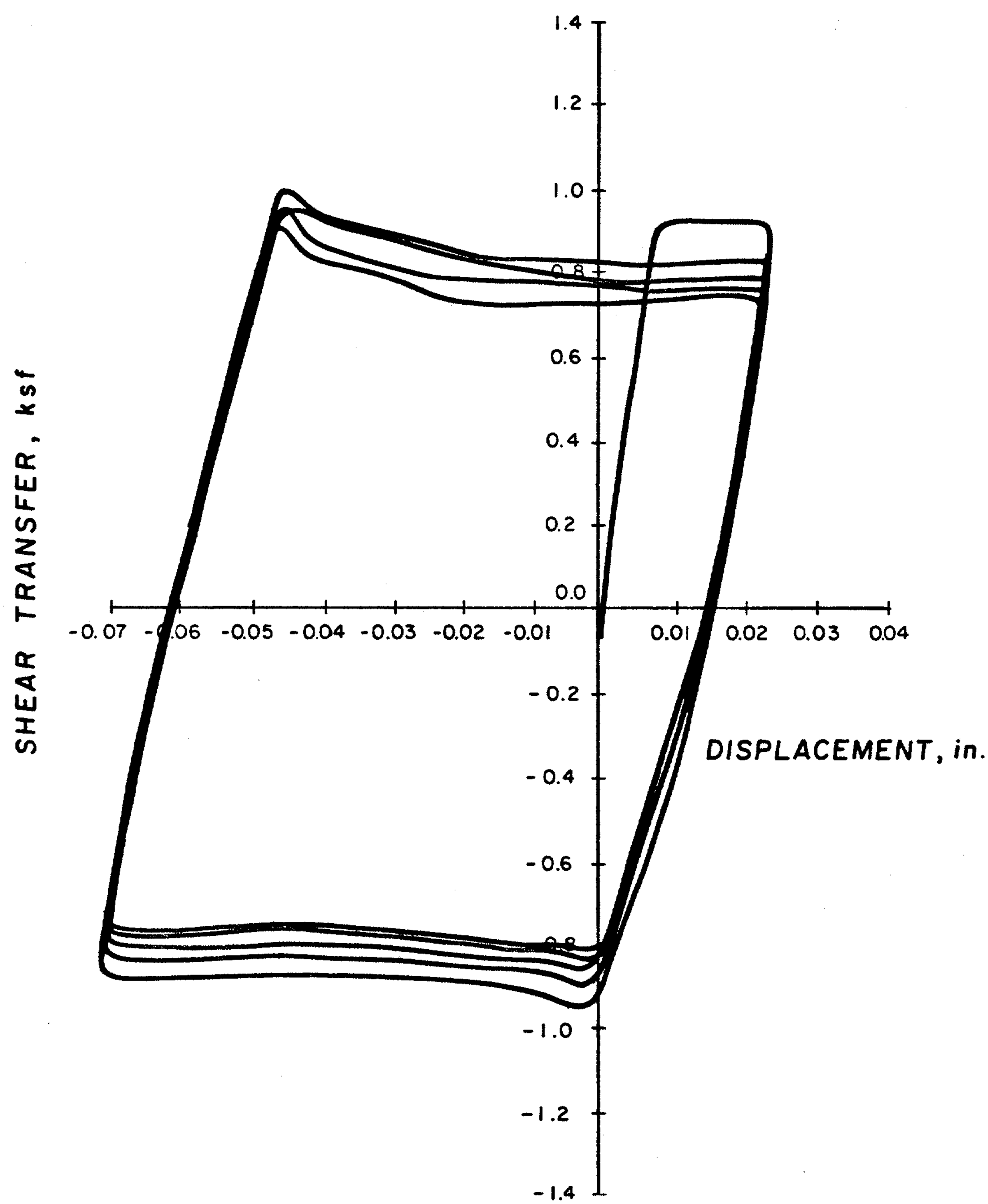
RESULTS OF THE INITIAL LOAD TEST AFTER CONSOLIDATION



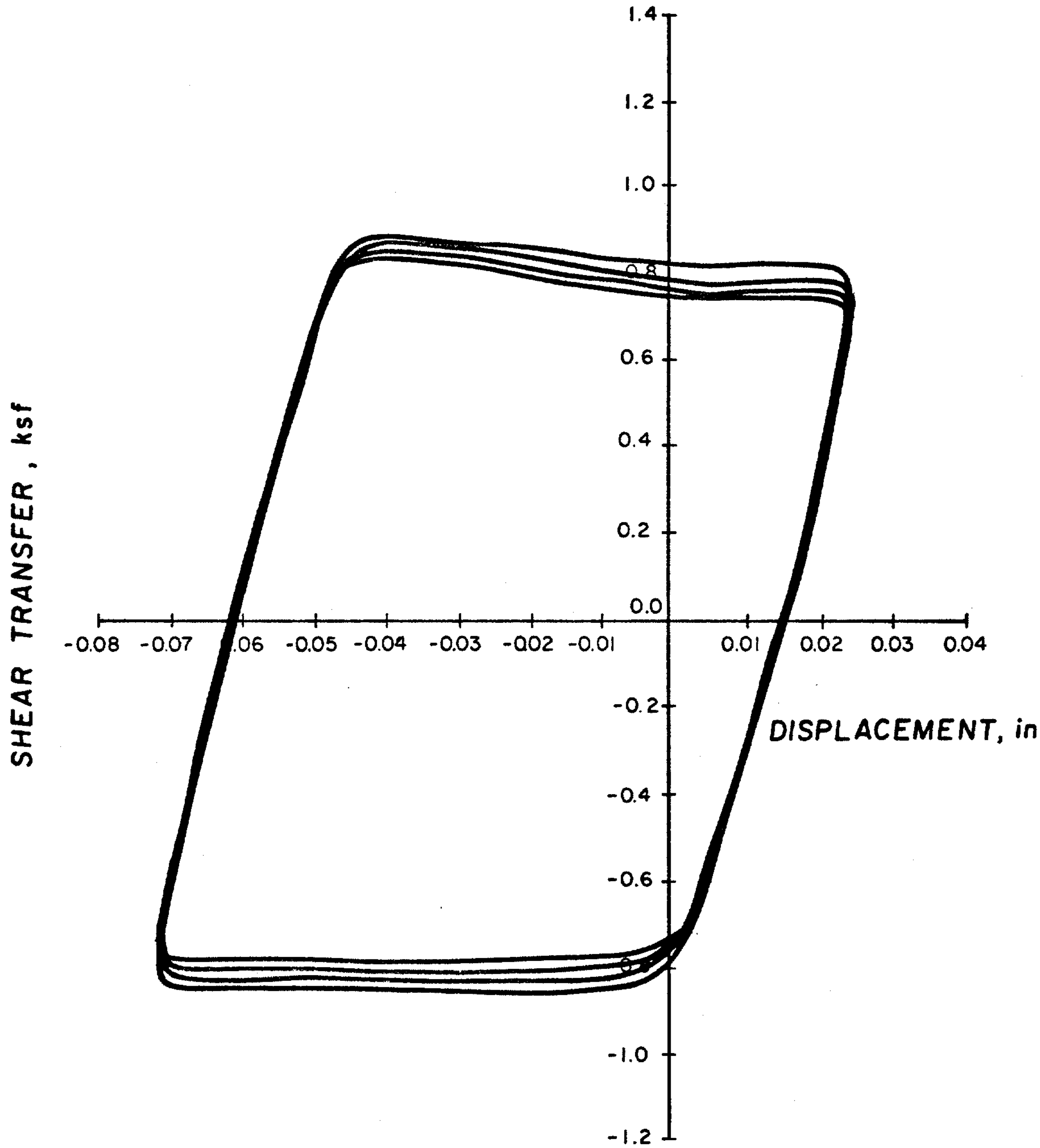
RESULTS OF ONE-WAY TENSION TESTS



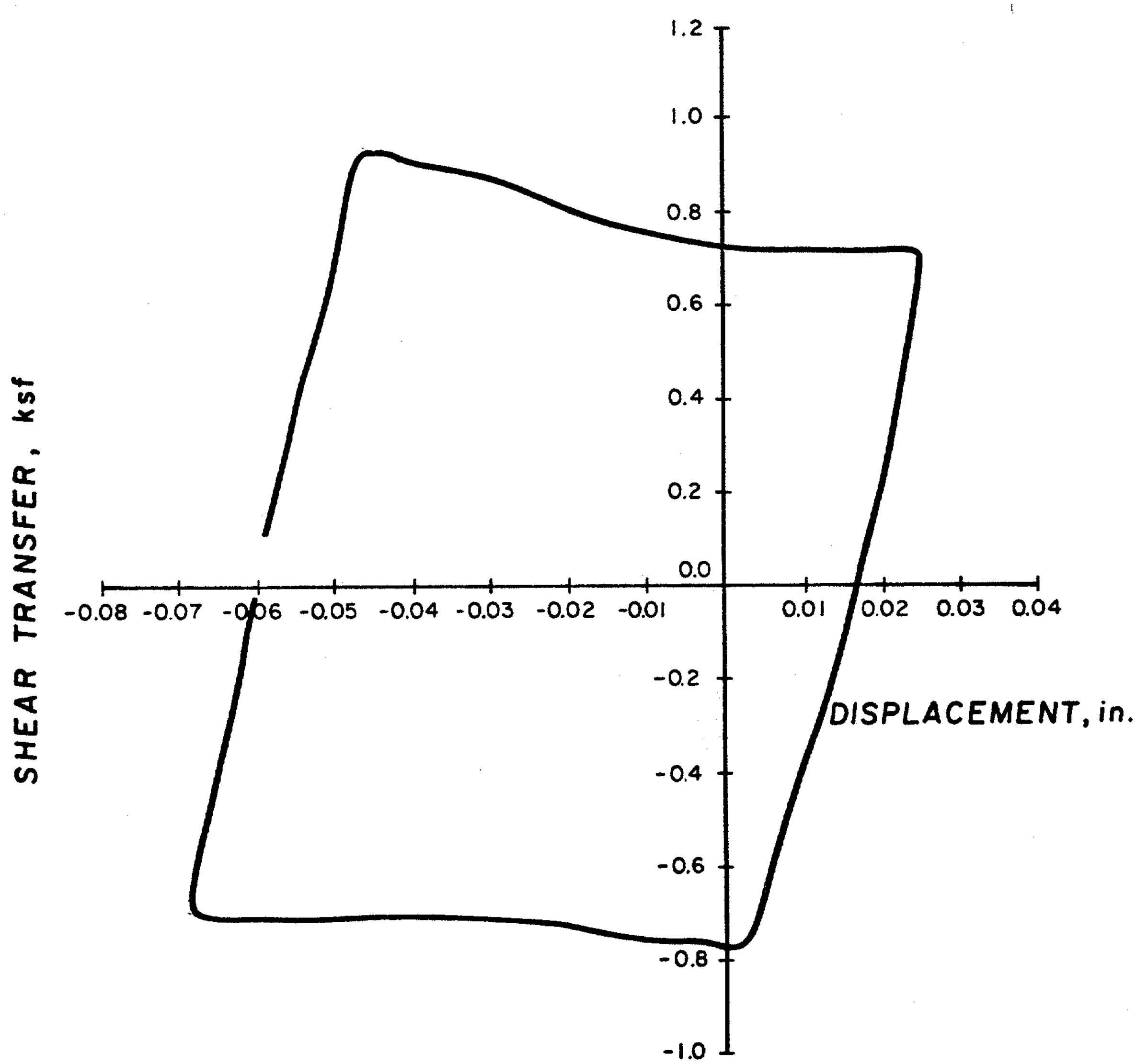
PRESSURE FLUCTUATIONS DURING ONE-WAY TENSION TESTS



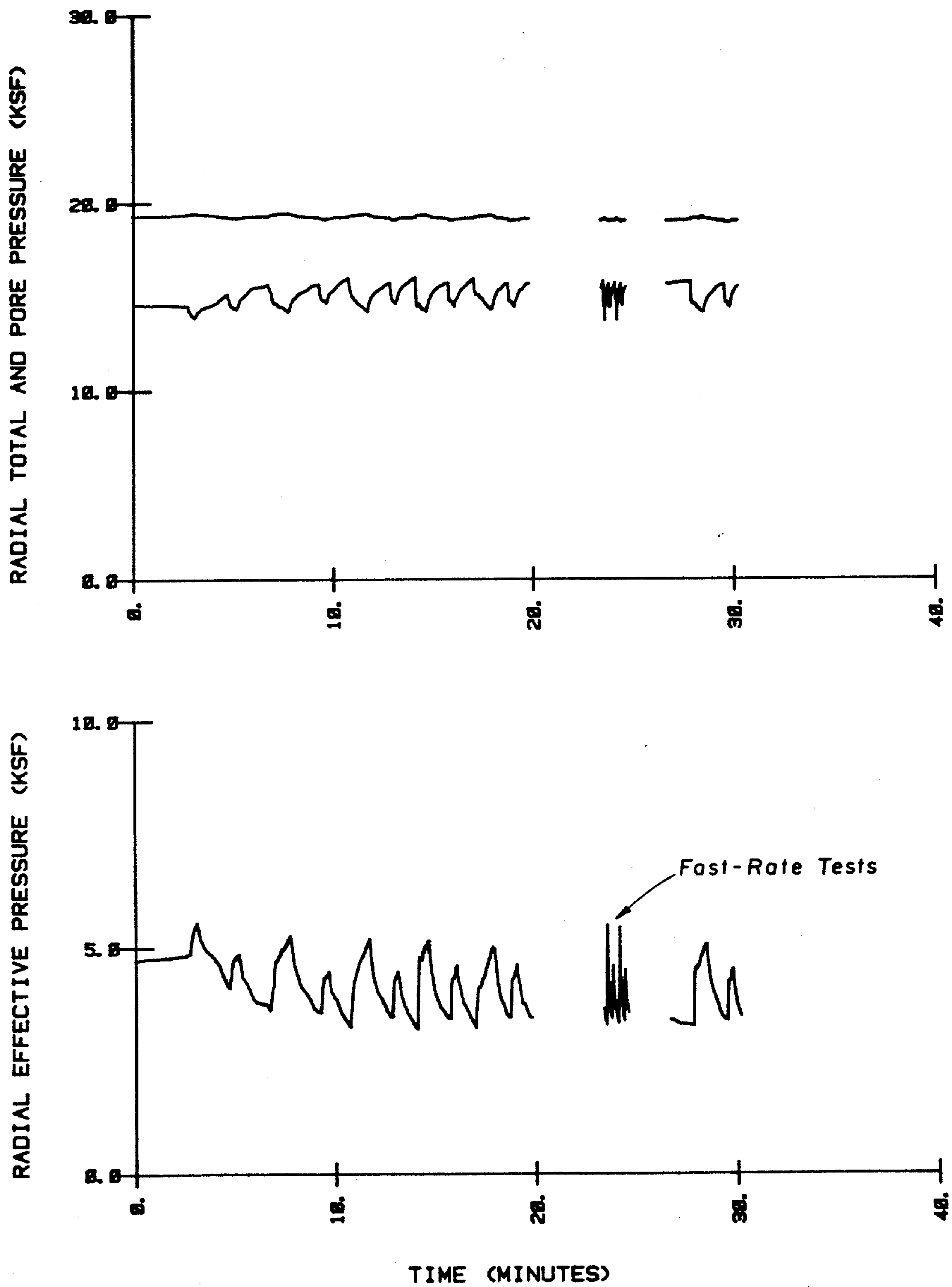
RESULTS OF THE TWO-WAY CYCLIC TEST



RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST

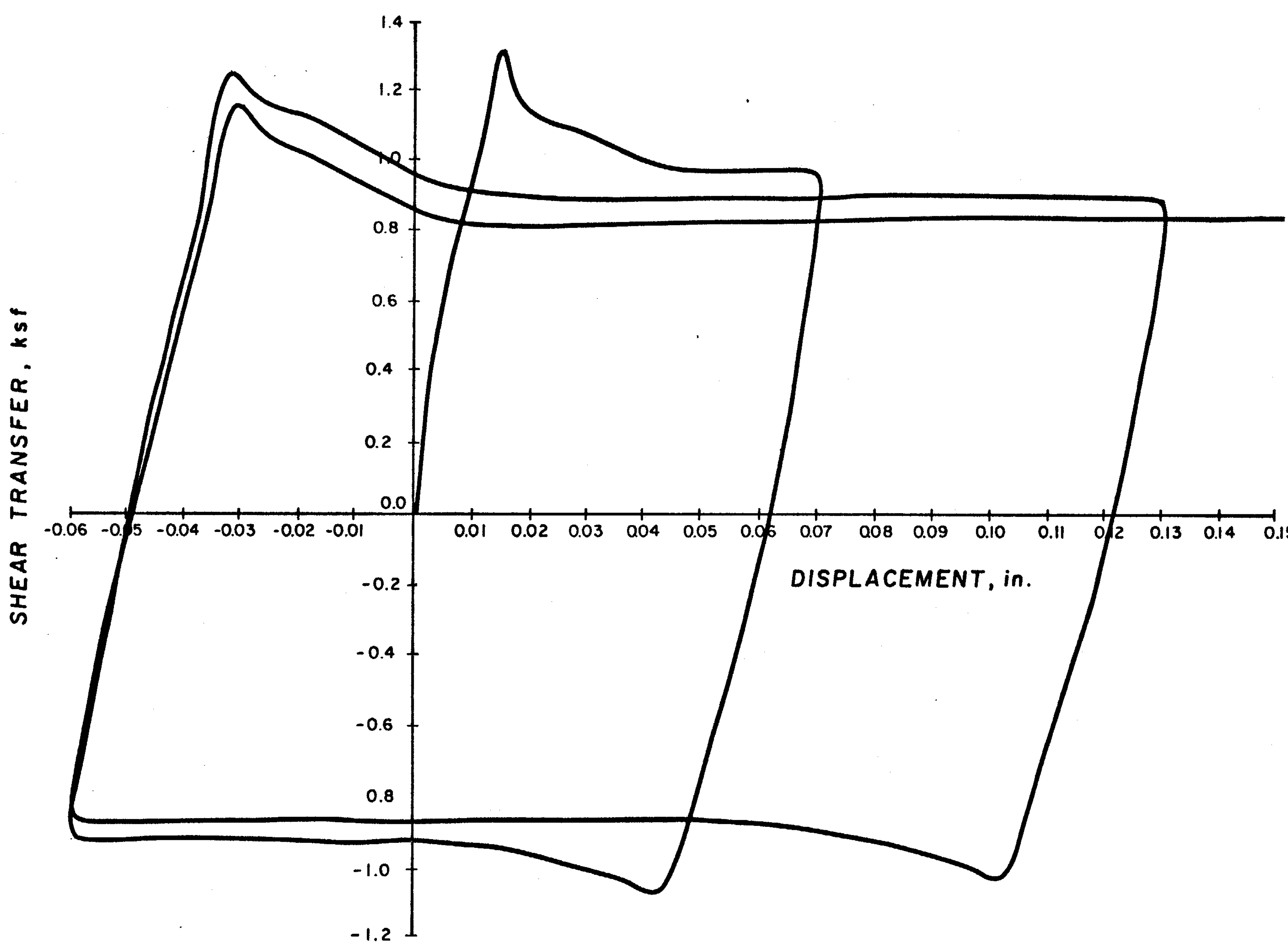


REPEAT OF THE SLOW-RATE TWO-WAY CYCLIC TEST



FLUCTUATIONS IN THE SOIL PRESSURES DURING TWO-WAY CYCLIC TESTS

JOB NO.__________



RESULTS OF THE LOAD TESTS AFTER ADDITIONAL CONSOLIDATION

APPENDIX D: SECOND X-PROBE EXPERIMENT AT THE 141-FT DEPTH

RESULTS OF THE SECOND X-PROBE EXPERIMENT AT THE 141-FT DEPTH

The second X-probe experiment at the 141-ft depth was performed during the period between 1000 hours on 22 April and 0800 hours on 23 April. The experiment was performed to investigate the repeatability of the results: Would two experiments at the same depth, but in borings 20 ft apart, yield the same results? As shown in the comparisons in Volume I, the answer was yes, if the soil is uniform horizontally.

The probe was installed using the drawdown on the drilling rig; the installation was completed on 22 April at 1014. The variations in the soil pressures during consolidation are shown in Plate D-1. The trends shown by the pressures are, again, a decrease in both the total and pore pressures with time, which combine to produce a linear trend in the increases in the radial effective pressure with the logarithm of time.

The maximum total pressure created by installation of the probe was 24.3 ksf, which had reduced to 22.6 ksf at the time of the first load test, 15 minutes after insertion. The maximum pore pressure after insertion was 21.9 ksf, which had decreased to 20.4 ksf at the time of the first load test.

The results of the first load test are shown in Plate D-2, which shows an initial peak shear transfer of 0.46 ksf, which reduced to a residual value of 0.37 ksf during continued slip after the load reversal.

In a manner similar to the first X-probe experiment at this depth, the soil was allowed to consolidate for approximately 4 hours, until 14:14, at which time the major series of load tests were performed. At this time, the total pressure had decreased to a value of 20.4 ksf; the pore pressure decreased to a value of 15.8 ksf, yielding a value of radial effective pressure of 4.6 ksf. Excellent agreement was obtained among the pressures measured during the first experiment at a similar time after installation, which were 20.2 ksf, 15.5 ksf, and 4.7 ksf, respectively.

Since this experiment was a repetition of one already performed, the one-way cylcic (repeated) load tests were omitted. The probe was loaded to failure in tension, then immediately subjected to a sequence of two-way, displacement-controlled cyclic loadings.

The results of the two-way cyclic tests are shown in Plate D-3. As shown in the plate, the maximum resistance on the first loading was 0.76 ksf, again showing excellent agreement with the experiment in the first boring, in which the shear transfer on the initial loading after consolidation was also 0.76 ksf. The peak resistance after load reversal was somewhat higher, being 0.84 ksf.

Five cycles of loading were applied. On the fifth cycle, the peak resistance was 0.74 ksf, with a residual shear of 0.64 ksf.

The rate of loading was then increased, with the results shown in Plate D-4. The shear transfer on the sixth cycle was 0.68 ksf.

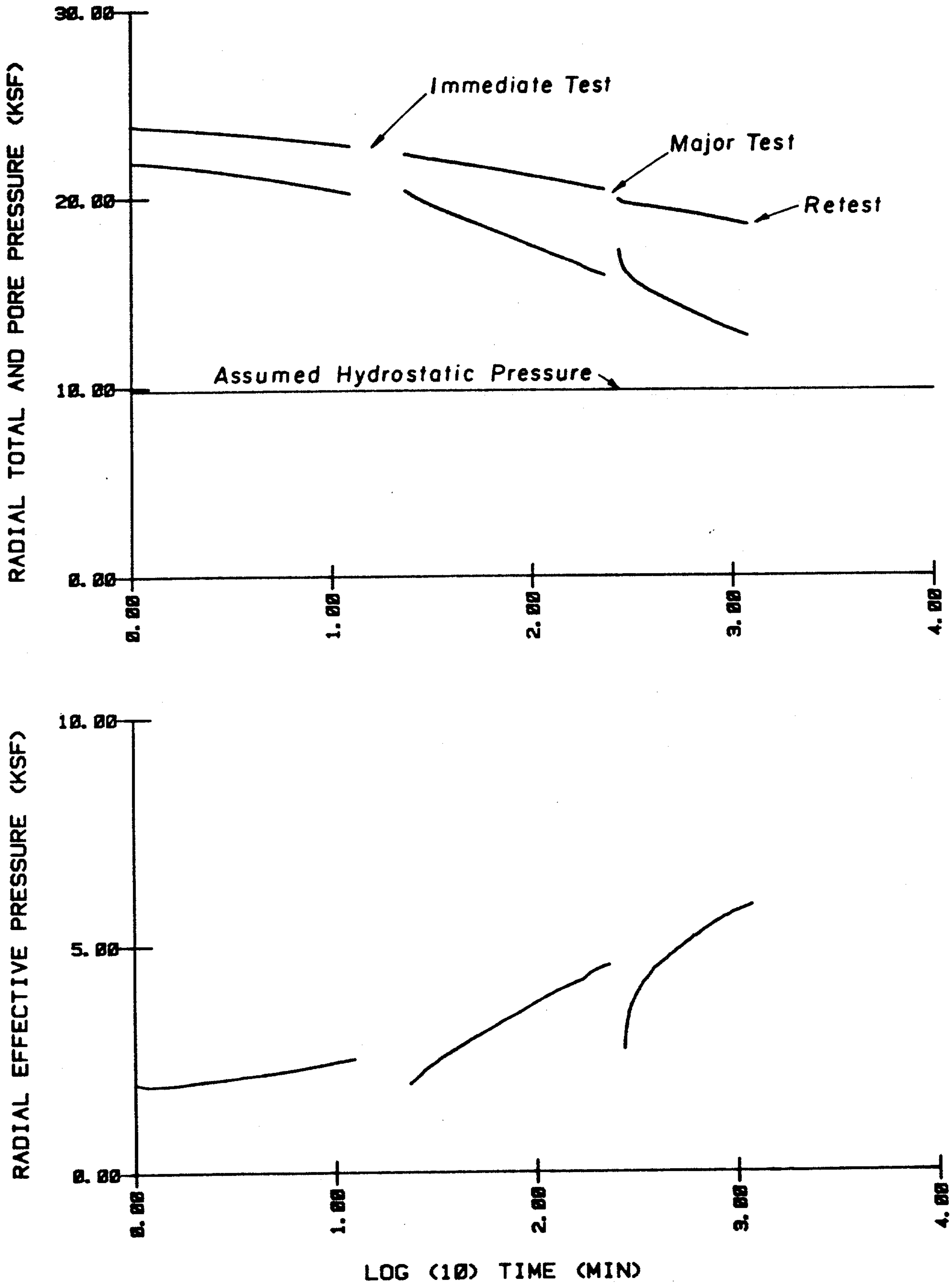
The loading rate was then decreased, with the results shown in Plate D-5. The peak resistance on the second cycle was 0.71 ksf; the residual shear was 0.60 ksf.

The fluctuations in the soil pressures during the cyclic tests are shown in Plate D-6. As seen in the figure, the initial loading to failure resulted in an increase in the pore pressure, a decrease in the effective pressure, with only minor effects on the total radial pressure. The pressures again fluctuated during the cyclic loading, with abrupt changes in the pore pressure and in the radial effective pressure accompanying the plastic slip. The soil pressures at the end of the cyclic load tests were a total radial pressure of 20.0 ksf, a pore pressure of 17.2 ksf, and a radial effective pressure of 2.8 ksf.

Upon the cessation of loading, the soil was allowed to consolidate for a period of approximately 16 1/2 hours prior to removal. During this period, the radial total pressure decreased to 18.5 ksf, the pore pressure decreased to 12.4 ksf, and thus the radial effective pressure increased to 6.1 ksf.

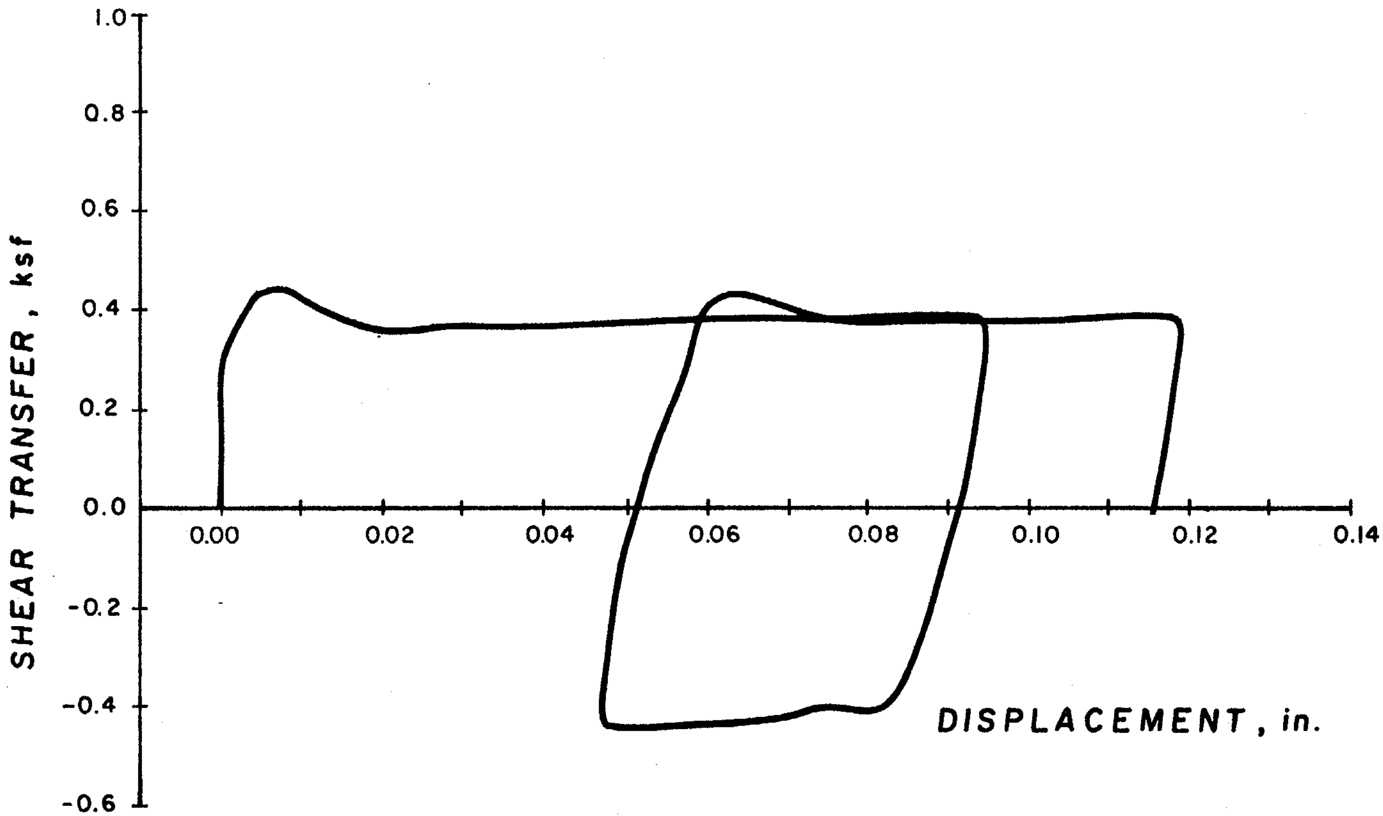
The probe was then subjected to a slow monotonic loading to failure in tension, one reversal to failure in compression, and then was pulled upward until the probe left the soil.

The results of this sequence of loading are given in Plate D-7. The peak shear on the first loading was 1.11 ksf; on the second failure in tension, 1.22 ksf; the residual shear after the second failure in tension was 0.95 ksf. Thus, the results of the X-probe experiments show good agreement, both with each other and with the estimated value of undrained shear strength at this depth of 1.25 ksf.

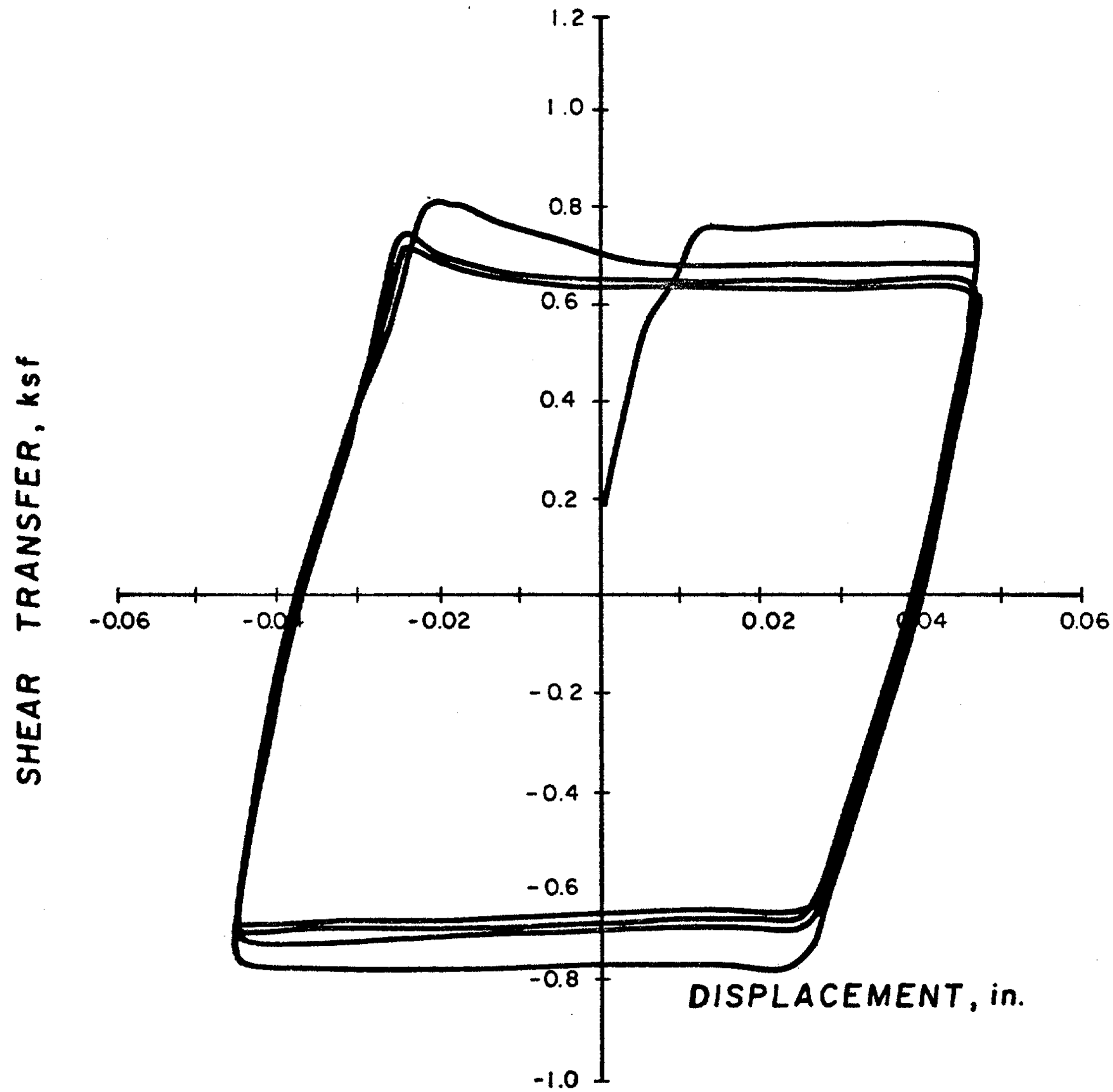


VARIATIONS IN SOIL PRESSURES DURING CONSOLIDATION

JOB NO.________

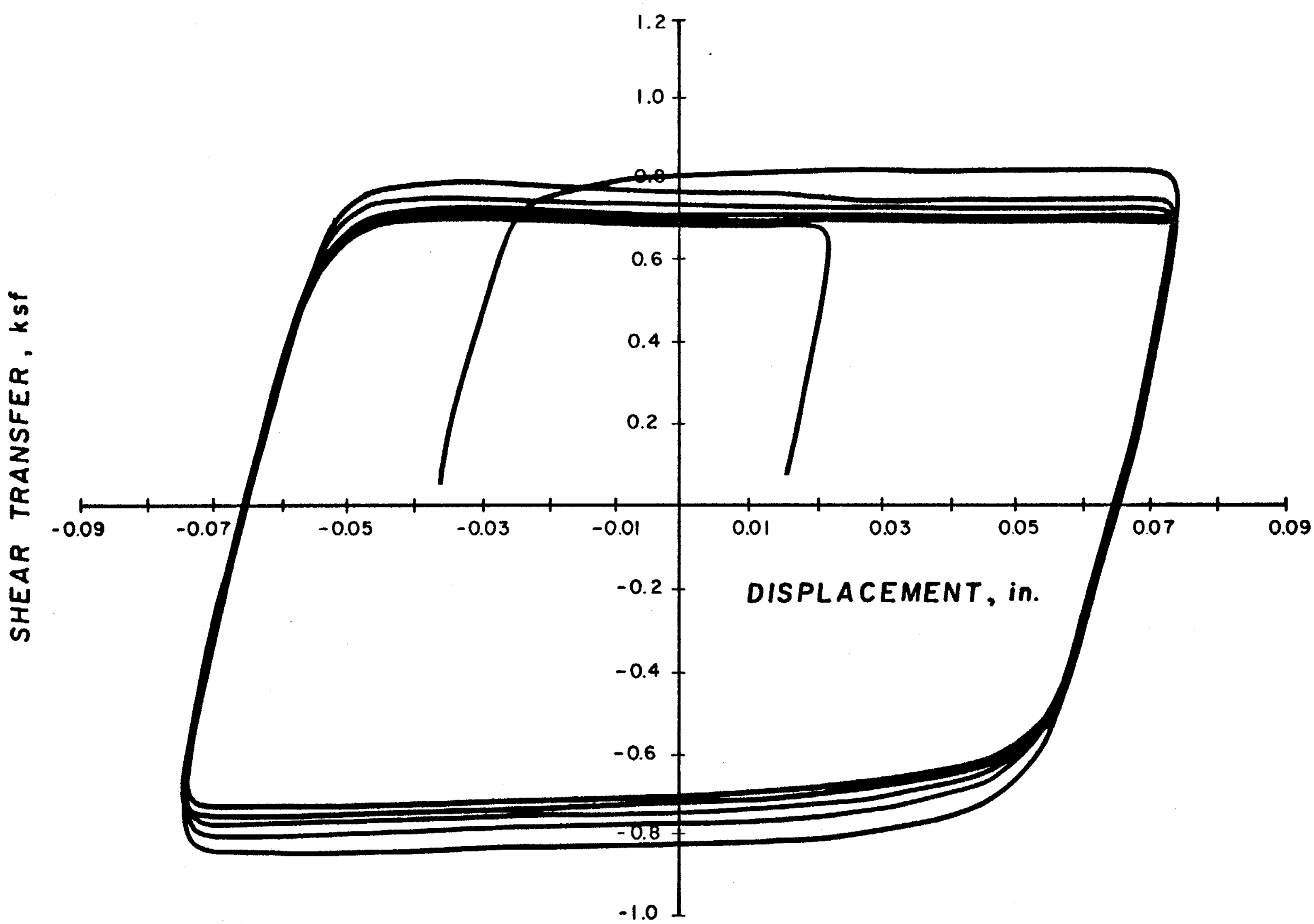


RESULTS OF THE LOAD TESTS IMMEDIATELY AFTER INSTALLATION

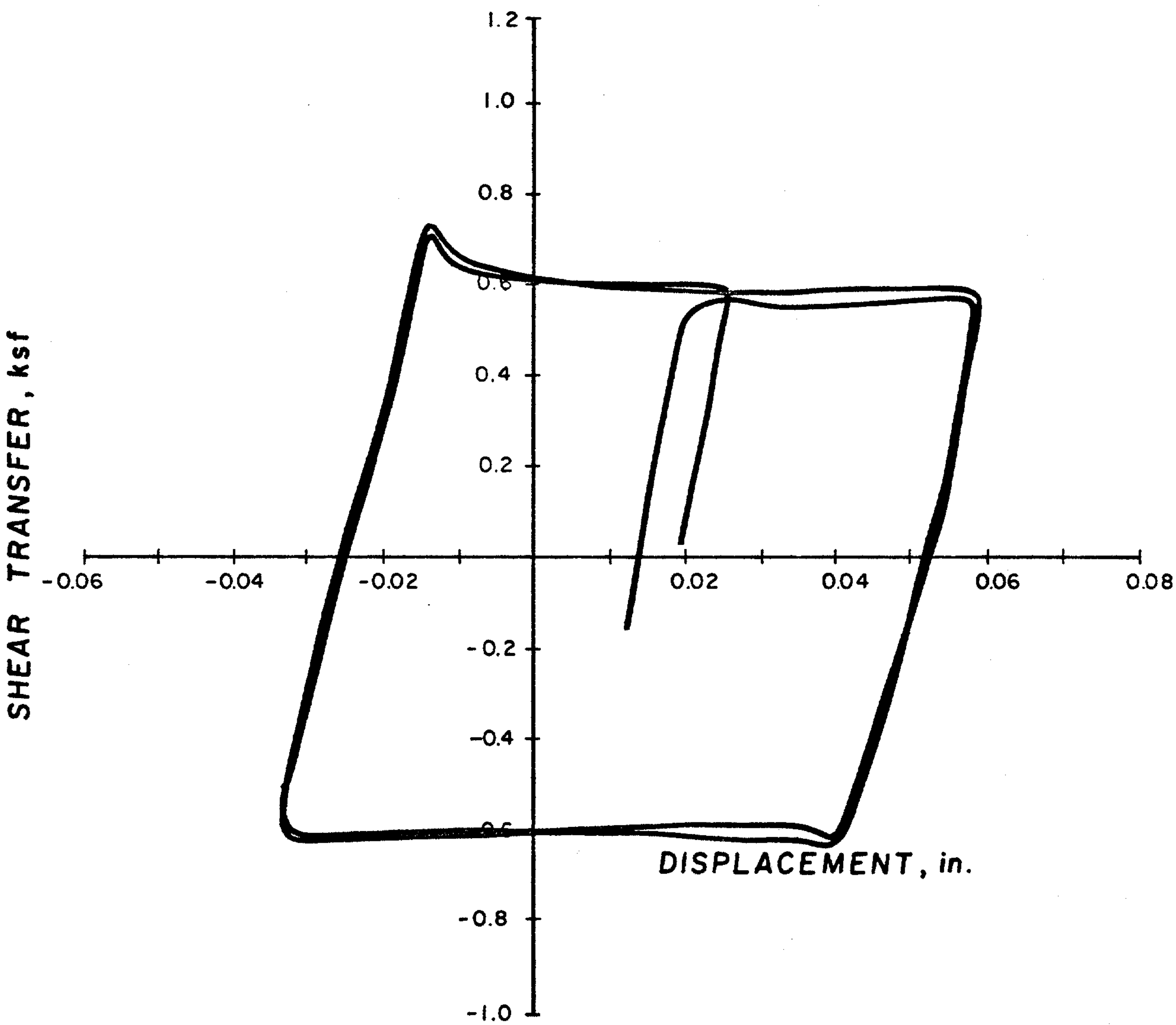


RESULTS OF THE TWO-WAY CYCLIC TEST

JOB NO. ______



RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST

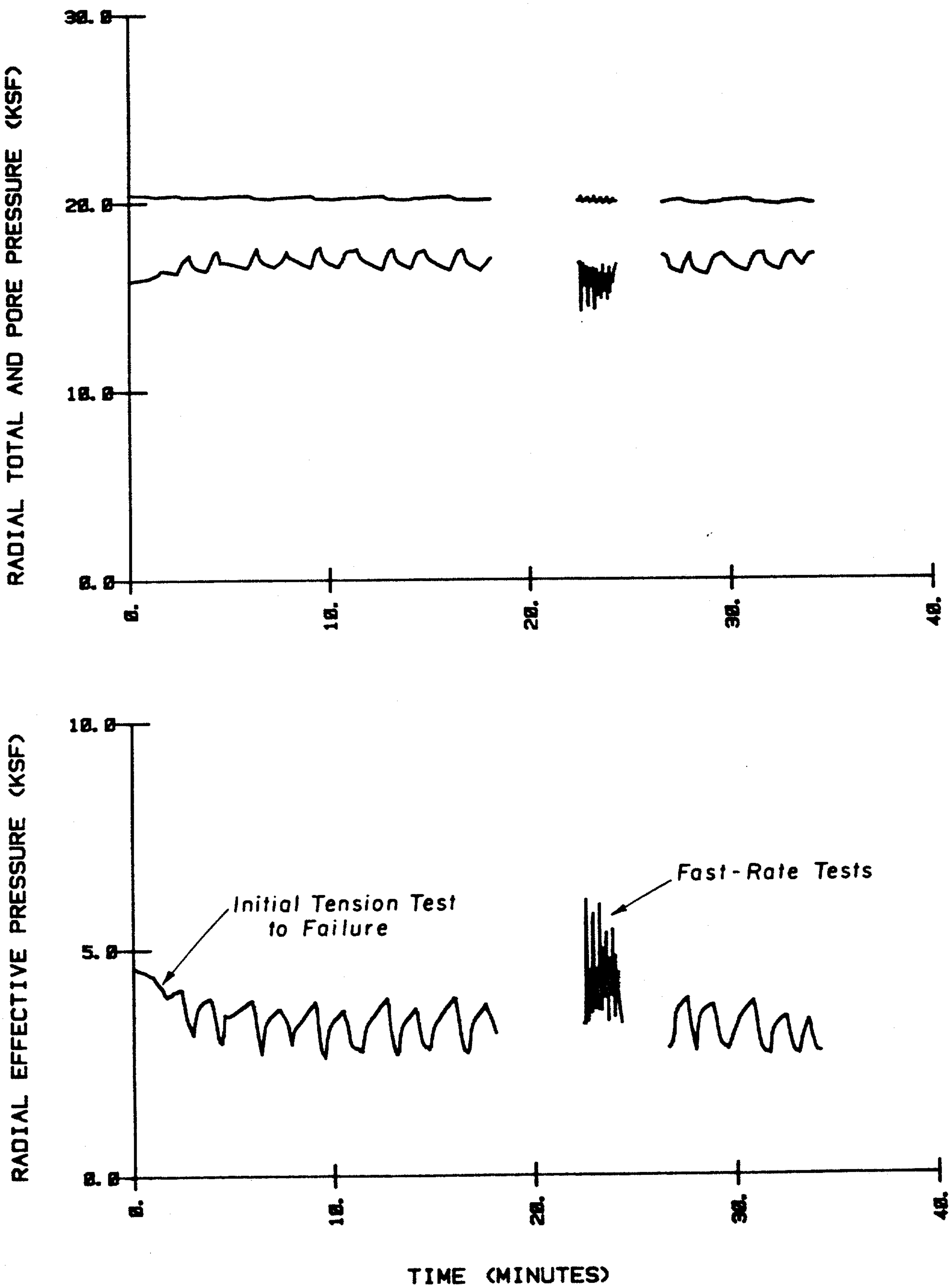


RESULTS OF THE REPEAT OF THE TWO-WAY CYCLIC TEST

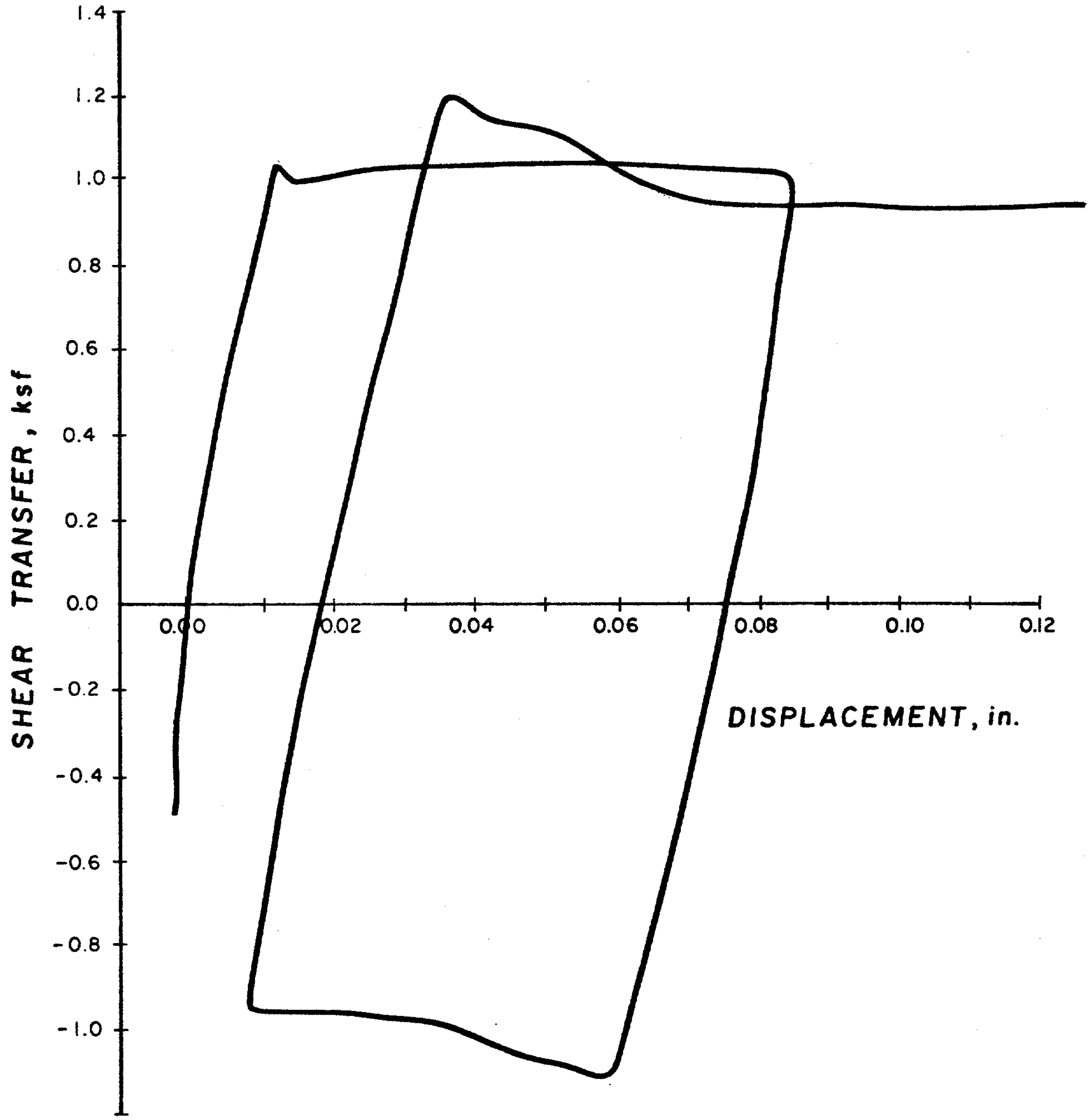
JOB NO.

The Earth Technology Corporation

JOB NO.



FLUCTUATIONS IN THE SOIL PRESSURES DURING TWO-WAY CYCLIC TESTS



RESULTS OF THE LOAD TESTS
PERFORMED AFTER ADDITIONAL CONSOLIDATION

APPENDIX E: CLOSED-END 3-IN. PROBE EXPERIMENT AT THE 141-DEPTH

RESULTS OF THE CLOSED-END 3-IN. PROBE EXPERIMENT AT THE 141-FT DEPTH

The experiment with the closed-end 3-in. probe was performed during the period from 1100 hours on 12 April to 0800 hours on 15 April.

The probe was driven into place with a 300-lb casing hammer, applying the impact force to the top of the N-rod string. Driving began at 11:12 and was completed at 11:26.

The maximum radial total pressure created by installation of the probe was 22.6 ksf; the accompanying pore pressure was 20.7 ksf. The subsequent changes in the soil pressures are given in Plate E-1. As shown in the plate, the consolidation was interrupted three times by load tests. It can be seen that, although the load tests create significant changes in the soil pressures, the soil tends to return to a consistent trend line, one which was established prior to the load tests.

As noted in Volume I, the load tests performed in this experiment included a load test 18 minutes after driving, a series of one-way cyclic (repeated) tension tests, and a series of two-way displacement-controlled cyclic tests, performed after a period of additional consolidation. The soil was then allowed to continue to consolidate for a period of 14 hours, but the probe was removed from the boring prematurely before a test could be performed.

The results of the test performed 18 minutes after driving are shown in Plate E-2. The peak shear transfer on the first failure was 0.38 ksf; the residual shear transfer after the second load reversal was 0.26 ksf. At the time the test began, the total radial pressure was 22.3 ksf; the pore pressure was 20.1 ksf; yielding a radial effective pressure of 2.2 ksf.

The initial loading to failure at the beginning of the first major series of load tests was begun at 00:52 on 13 April, 12 1/2 hours after driving. At this time, the total radial pressure had decreased to 20.4 ksf, and the pore pressure had decreased to 15.5 ksf, yielding a radial effective pressure of 4.9 ksf. A comparison of these values of pressure with those obtained during the two X-probe experiments shows excellent agreement.

The results of the initial loading to failure after consolidation are shown in Plate E-3. The peak shear transfer in the figure is 0.76 ksf, also indicating excellent agreement with the two X-probe experiments. The residual shear shown in the figure is 0.67 ksf.

Following the initial loading to failure, the probe was unloaded to near-zero shear, and the pore pressures were monitored for 43 minutes, during which time the total pressure increased slightly, from 20.1 to 20.2 ksf, the pore pressure decreased from 16.3 to 15.8 ksf, and the radial effective pressure increased from 3.8 to 4.4 ksf.

The sequence of one-way cyclic tension loadings were then applied, with the results summarized in Plate E-4. As shown in the plate, the accumulation of progressive upward displacements began at peak load levels greater than about 60 percent of the initial failure load, with rapid progressive pullout occurring when the load level was increased to a level greater than the value of the maximum resistance on the initial loading.

Again, the linear accumulation of permanent displacement at repeated load levels less than the static maximum are a consequence of the conditions of the test: the probe is a short, stiff member. Due to the nonlinear, inelastic t-z response, such behavior should be expected. Again, the rate of accumulation of permanent displacement is linear, indicating that no degradation is occurring in the soil. Had any cyclic degradation accompanied the repeated loading, the rate of displacement should have increased with the number of cycles.

The fluctuations in the soil pressures during the one-way repeated tension tests are shown in Plate E-5. Unlike the behavior observed in the X-probe experiments, the pressure fluctuations are due to changes in the radial effective pressure; only very small effects are noted in the pore pressure. Attempts were then made to apply a series of two-way displacement-controlled cycles. Upon loading the probe in compression, the earth anchors which were used to provide the reaction force were pulled from the ground. The loading was therefore stopped, and additional earth anchors were obtained.

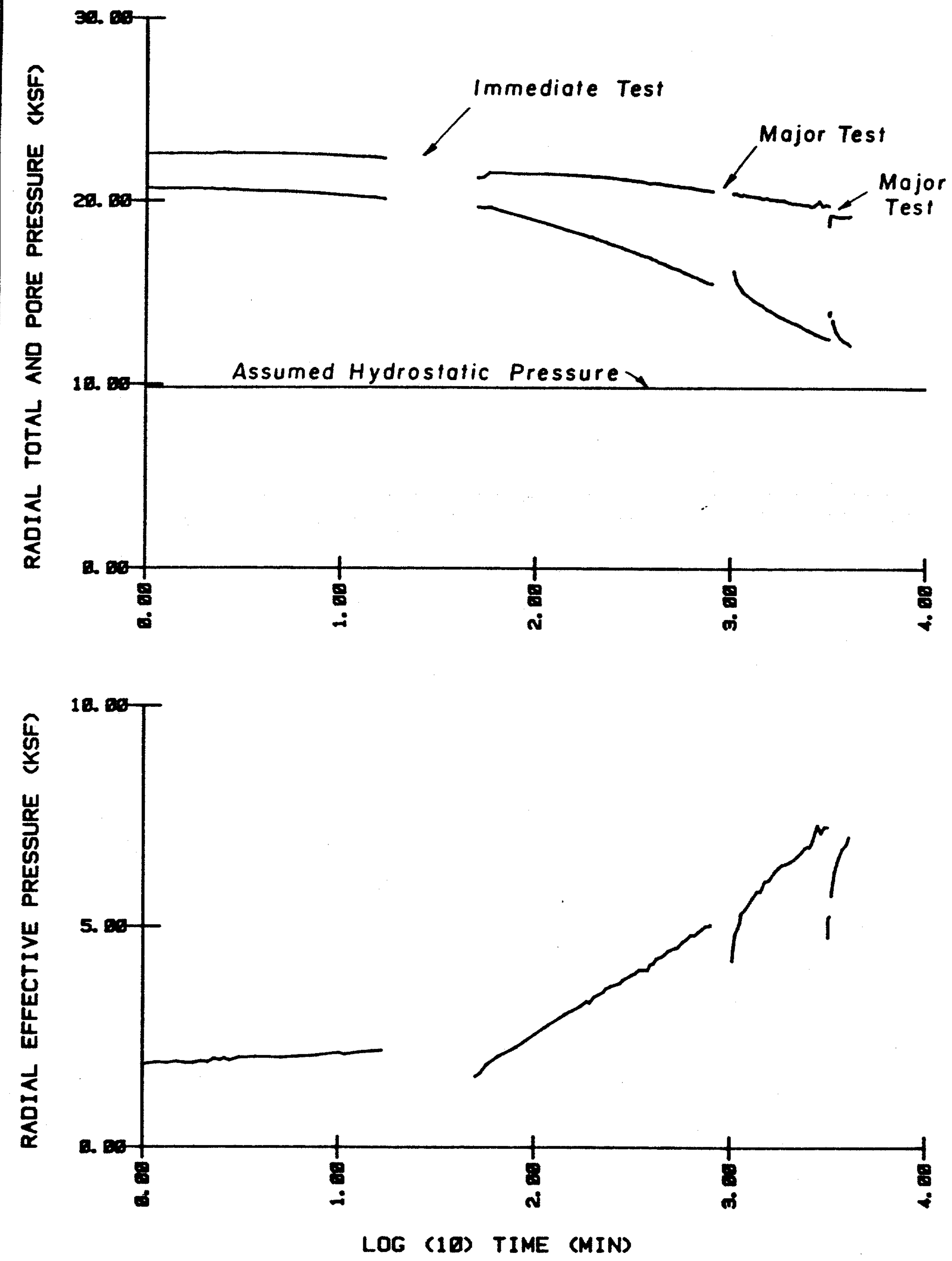
The additional consolidation which occurred during this period is shown in Plate E-1. The radial total pressure decreased from a value of 20.4 ksf at the end of the cyclic tension tests to a value of 19.8 ksf prior to the next loading. The pore pressure decreased from 16.2 ksf to 12.5 ksf; yielding an increase in the radial effective pressure from 4.2 ksf to 7.3 ksf.

The probe was then subjected to a slow monotonic loading to failure in tension and then to a 10 cycles of two-way displacement-controlled loading to failure in tension and compression, with the results shown in Plate E-6. The maximum shear transfer on the first loading was 1.21 ksf; after 10 cycles, the peak shear tranfer had decreased to 0.87 ksf, with a residual shear transfer of 0.82 ksf.

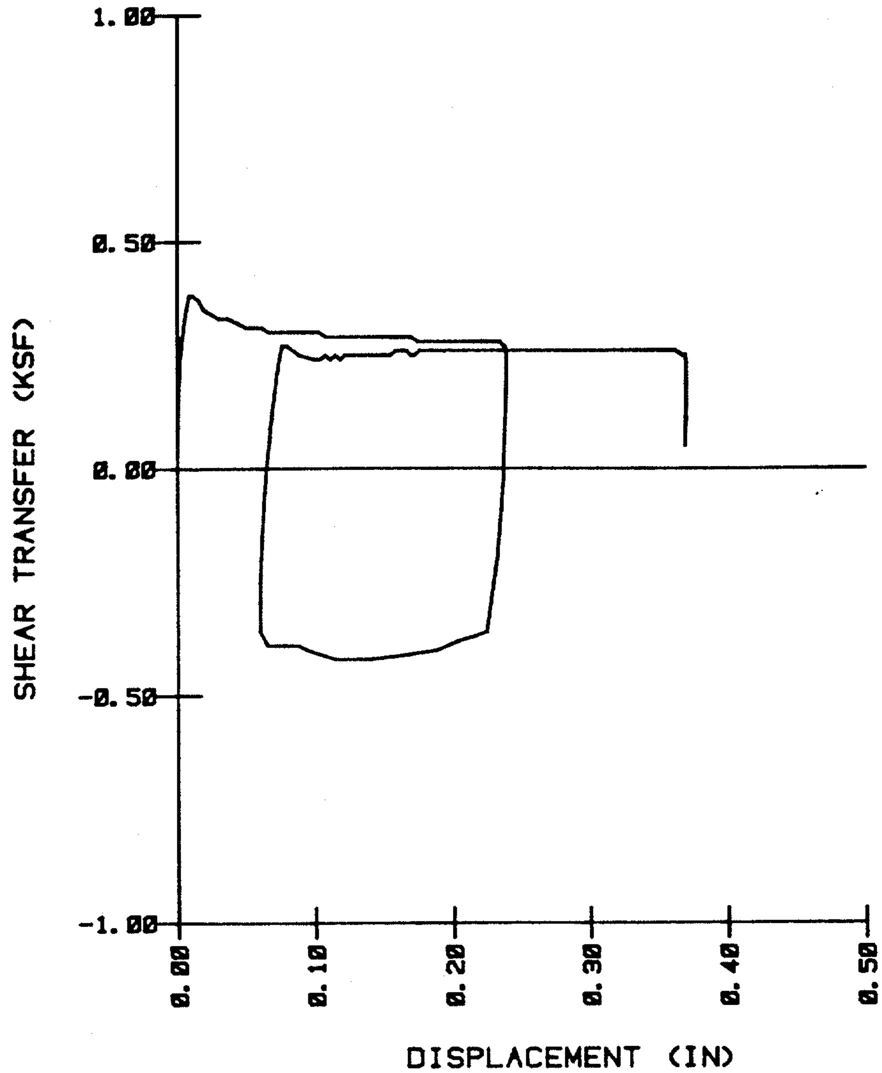
The rate of loading was then increased, with the results shown in Plate E-7. On the fourth cycle, the shear transfer was 0.97 ksf, with the rate of slip being 0.0203 in./sec.

The rate of loading was then decreased, with the results shown in Plate E-8. The peak resistance decreased to 0.81 ksf and the residual shear to 0.76 ksf. The rate of displacement during these loadings was 0.0013 in./sec. The effect of rate on the limiting shear was thus 23 percent per log cycle of increase in the rate of slip.

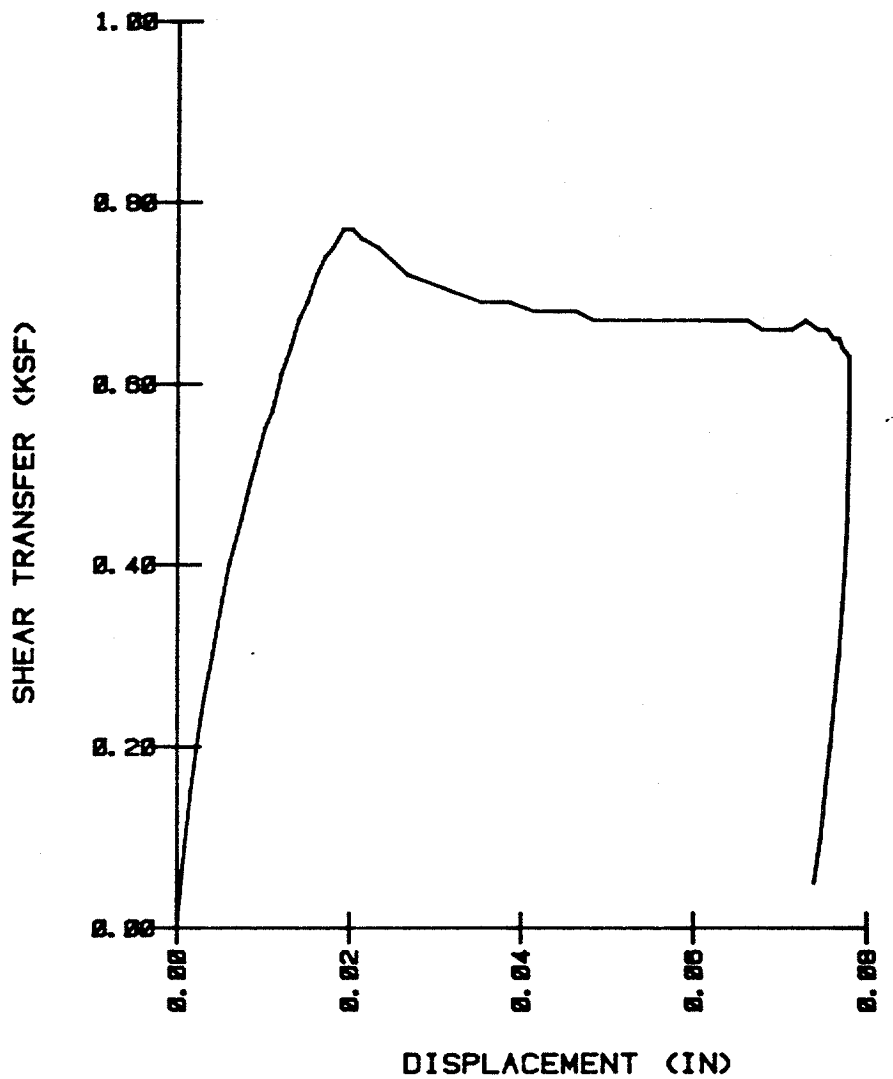
The fluctuations in pressure during the two-way cyclic tests are shown in Plate E-9. The changes during the initial loading to failure are shown in the left-most portion of the figure; the testing was stopped for about 15 minutes to check the earth anchors and to tighten the connections. It can be seen that the fluctuations follow the same pattern as shown earlier for the X-probes; the effective pressure increases significantly during plastic shear. During these tests, however, the increases are not due to pore pressure changes alone.



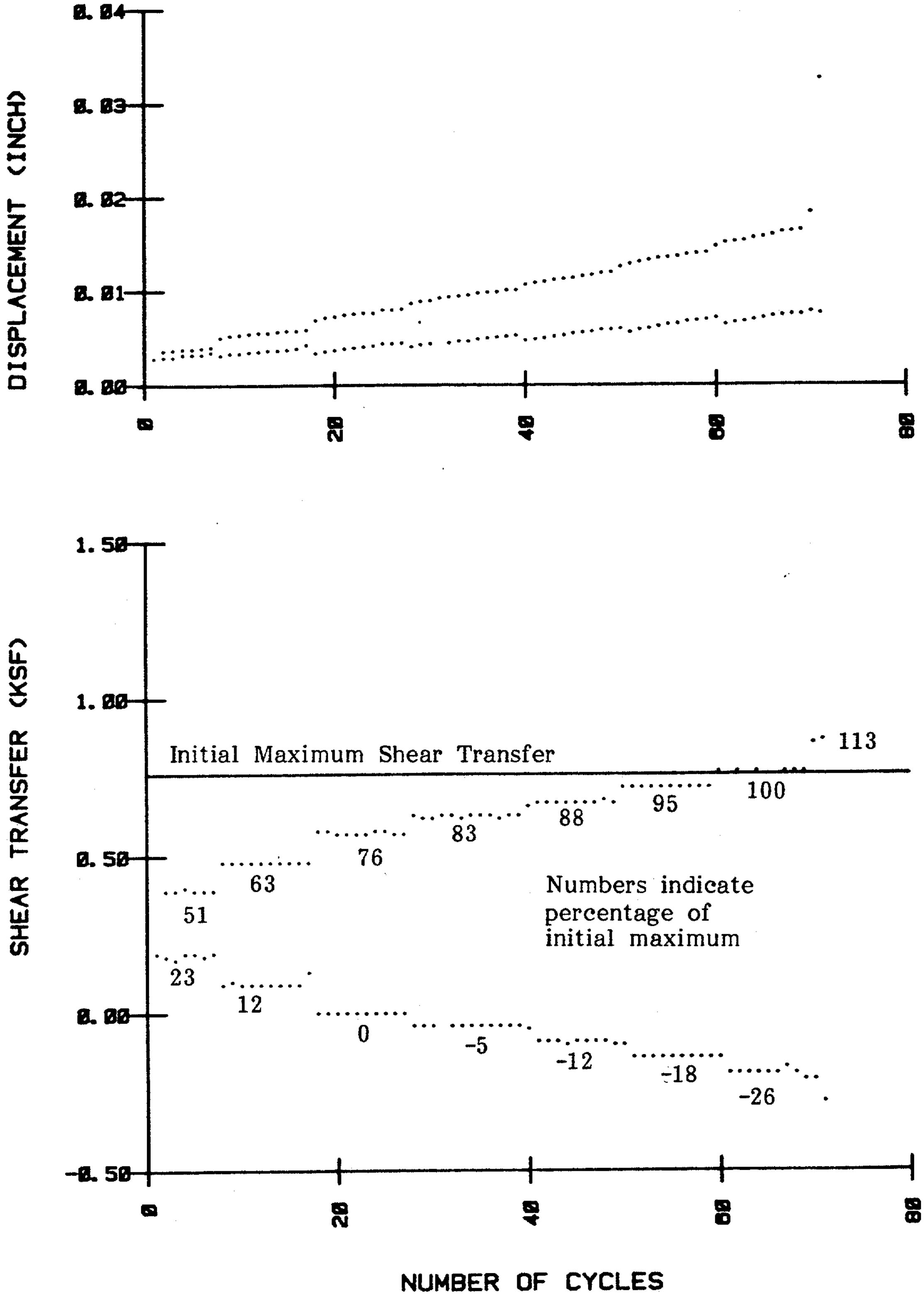
VARIATION IN SOIL PRESSURES DURING CONSOLIDATION



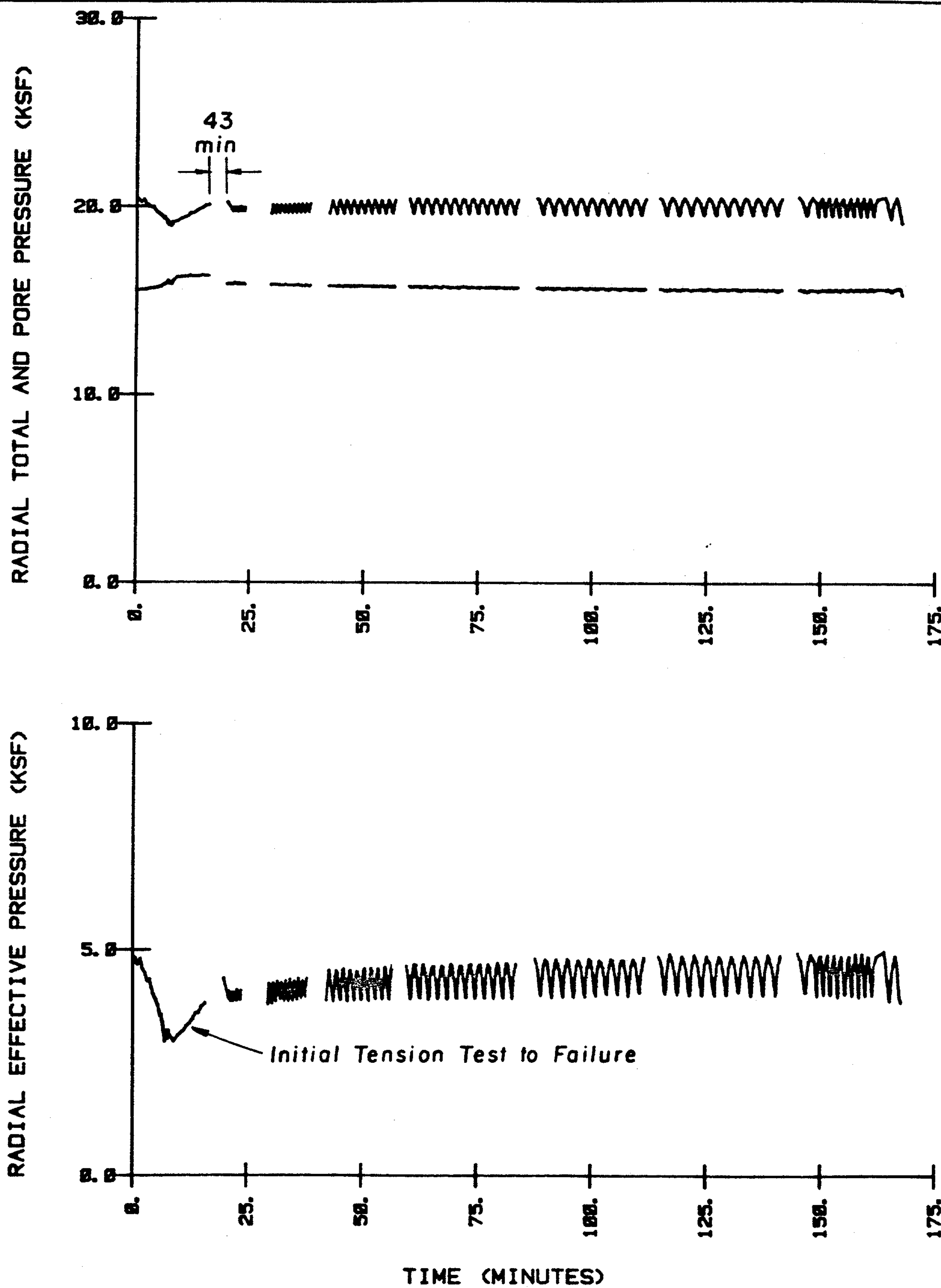
RESULTS OF THE LOAD TEST IMMEDIATELY AFTER DRIVING



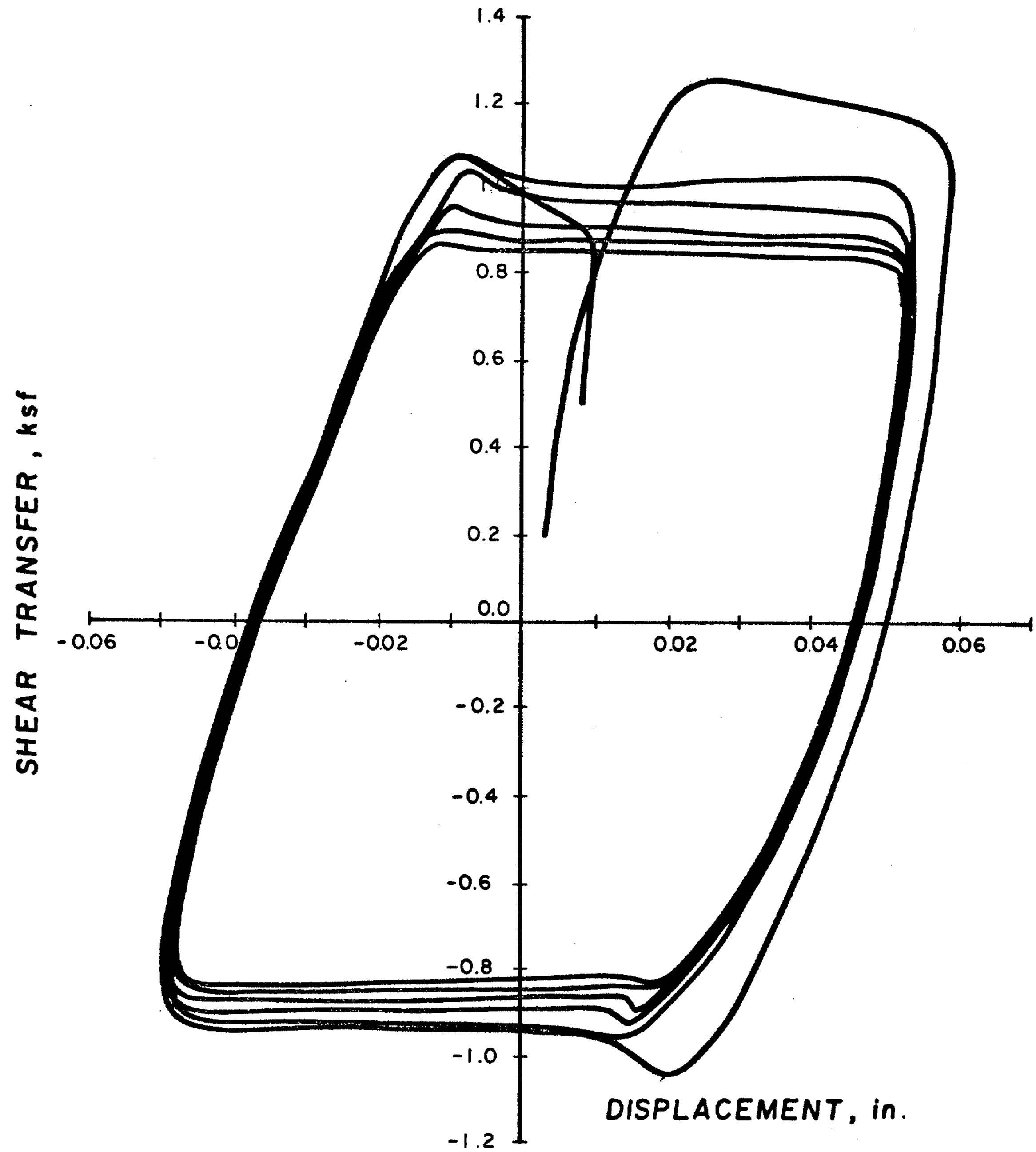
RESULTS OF THE INITIAL LOAD TEST AFTER CONSOLIDATION



RESULTS OF ONE-WAY CYCLIC TENSION TESTS



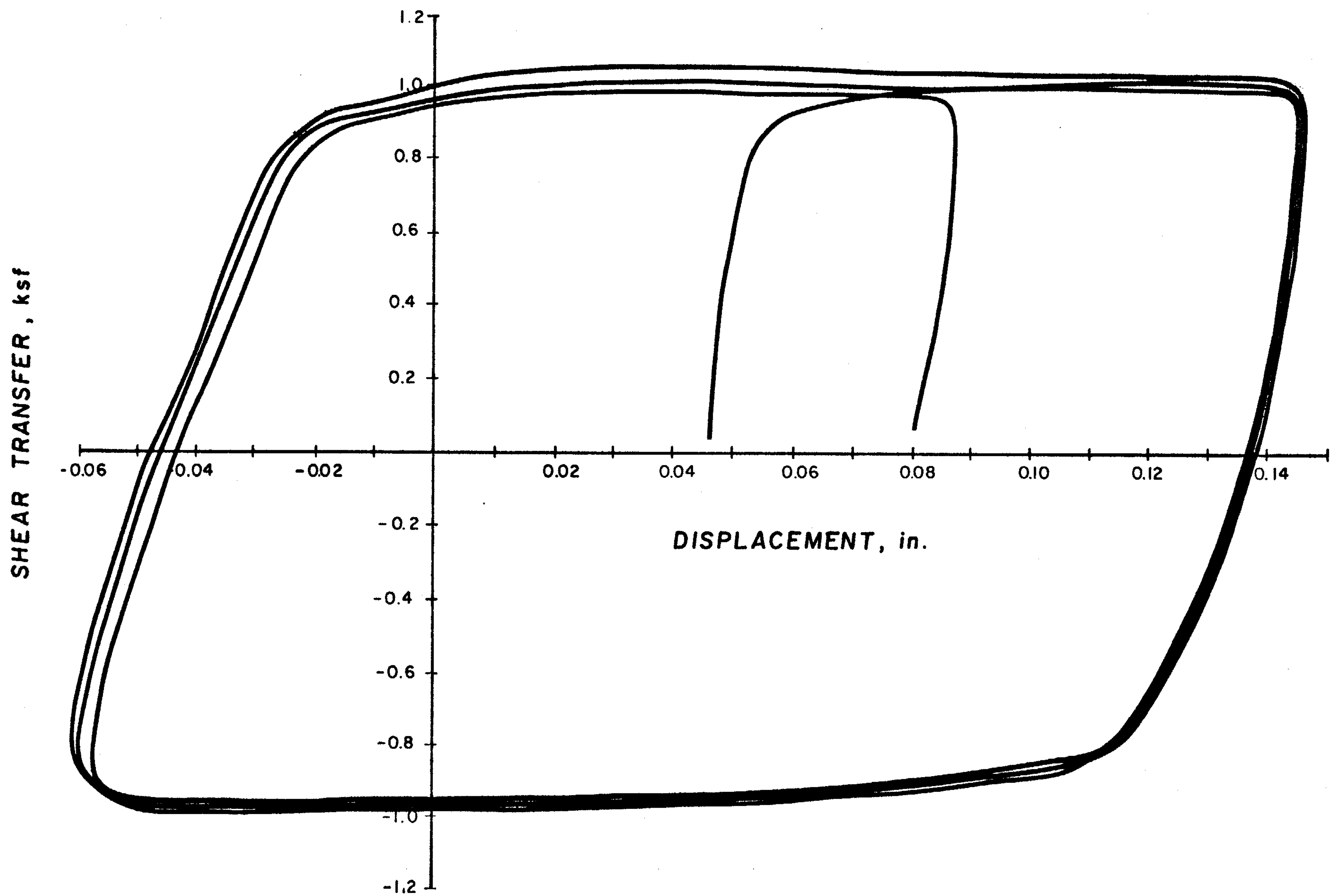
FLUCTUATIONS IN SOIL PRESSURE DURING ONE-WAY CYCLIC TENSION TESTS



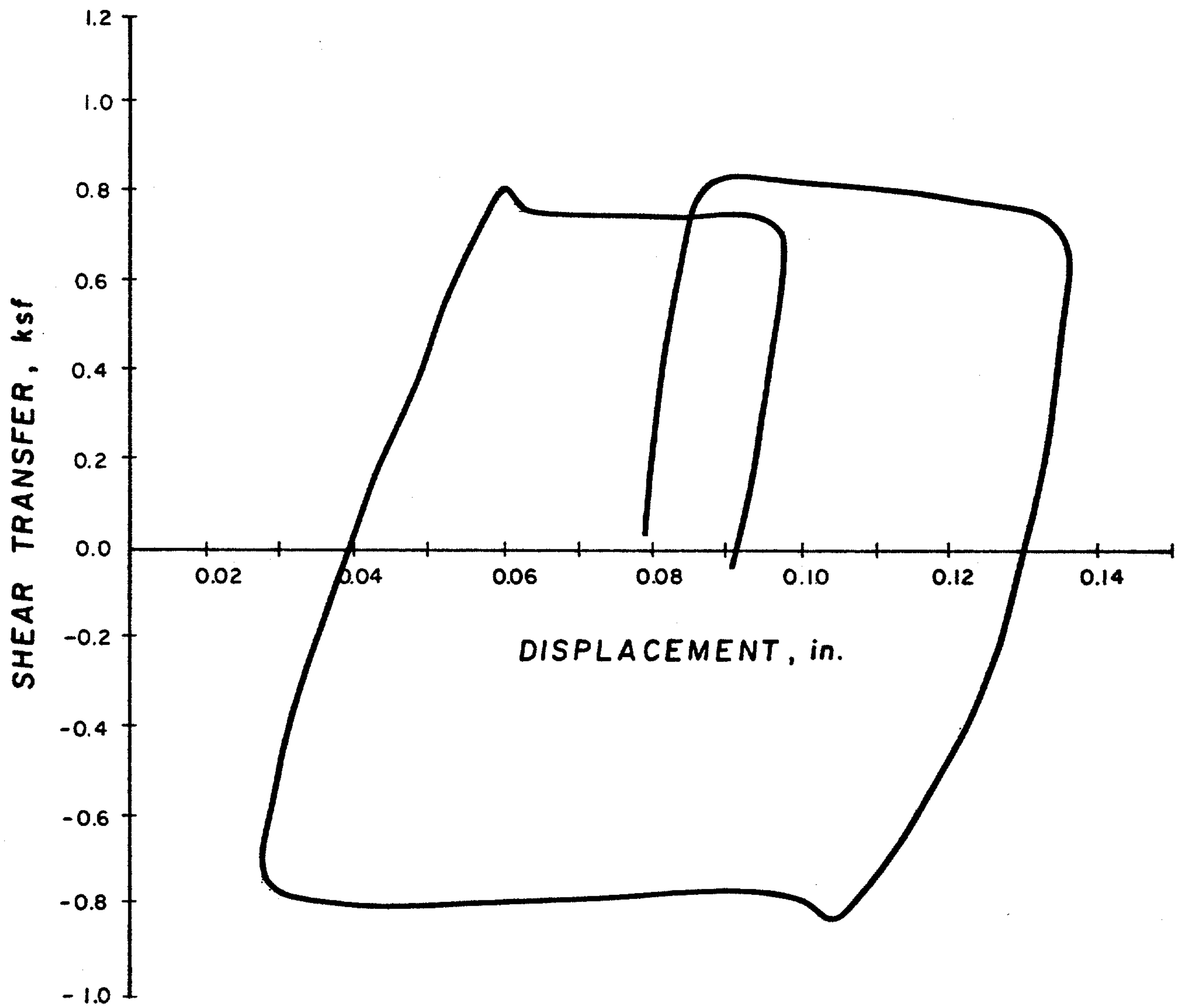
RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST

The Earth Technology Corporation

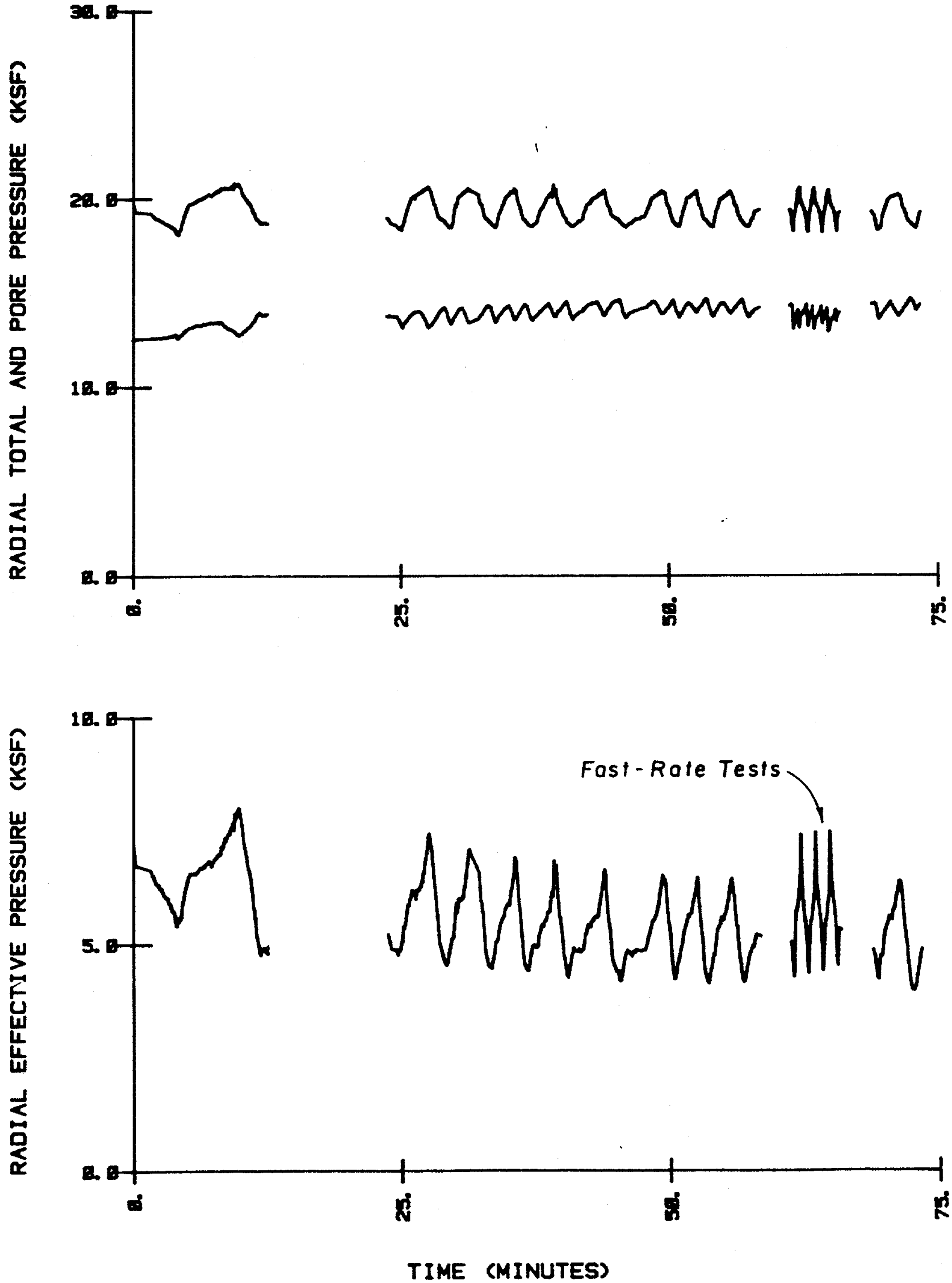
JOB NO.



RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST



RESULTS OF THE REPEAT OF THE TWO-WAY CYCLIC TEST



FLUCTUATIONS IN SOIL PRESSURE DURING TWO-WAY CYCLIC LOADING

APPENDIX F: OPEN-END 3-IN. PROBE EXPERIMENT AT THE 141-FT DEPTH

RESULTS OF THE OPEN-END 3-IN. PROBE EXPERIMENTS AT THE 141-FT DEPTH

The experiment with the open-end 3-in. probe at the 141-ft depth was performed during the period from 1130 hours on 10 April until 0800 hours on 11 April.

The probe was driven into place using a 900-lb casing hammer, applying the impact force to the top of the N-rod string. The driving was begun at 11:28 and was completed at 11:44.

The variations in soil pressure after installation of the probe are shown in F-1. In contrast with the behavior exhibited by the full-displacement probes, the value of radial effective pressure immediately after driving was very small; this indicates that not much cavity expansion occurred during the installation. Low values of initial radial effective pressure are to be expected from such a probe (or pile), since the value of initial effective pressure varies considerably with changes in the volume of soil which is forced outward during installation.

As shown in the plate, a load test was performed a few minutes after driving. At the time of the test, the total radial pressure had decreased from a maximum value of 18.4 ksf to 17.7 ksf; the pore pressure had decreased from a maximum value of 18.1 ksf to 16.7 ksf, yielding an increase in the radial effective pressure from 0.3 ksf to 1.0 ksf.

The results of the first load test are given in Plate F-2. The maximum shear on the first loading was 0.59 ksf; the residual shear on the second tension failure was 0.42 ksf.

The soil was allowed to consolidate for 12 hours prior to performing the major series of load tests. During this period, the total pressure decreased from a value at the end of the first load test of 17.4 ksf to 15.3 ksf the pore pressure decreased from 15.9 ksf to 10.7 ksf, yielding an increase in the radial effective pressure from 1.5 ksf to 4.6 ksf.

Since this was the first experiment to be performed at the site, the cyclic (repeated) tension tests were not performed. The results of the initial slow monotonic loading to failure in tension are given in Plate F-3. The initial maximum shear transfer was 1.04 ksf; the residual shear transfer was 0.90 ksf.

Upon completion of the test, the radial total pressure was 15.1 ksf; the pore pressure was 11.2 ksf, yielding a value of 3.9 ksf for the radial effective pressure. The initial loading to failure thus resulted in a net decrease in the radial effective pressure of 0.7 ksf.

The probe was then subjected to five cycles of two-way loading, the first three of which are shown in Plate F-4. The shear transfer on the first loading to failure was 1.08 ksf; on the fifth cycle, the peak shear was 0.93 ksf, with a residual shear transfer of 0.85 ksf.

The load rate was then increased, as shown inPlate F-5. On the fifth cycle, the limiting shear transfer was 1.09 ksf, at a slip rate of 0.0135 in./sec.

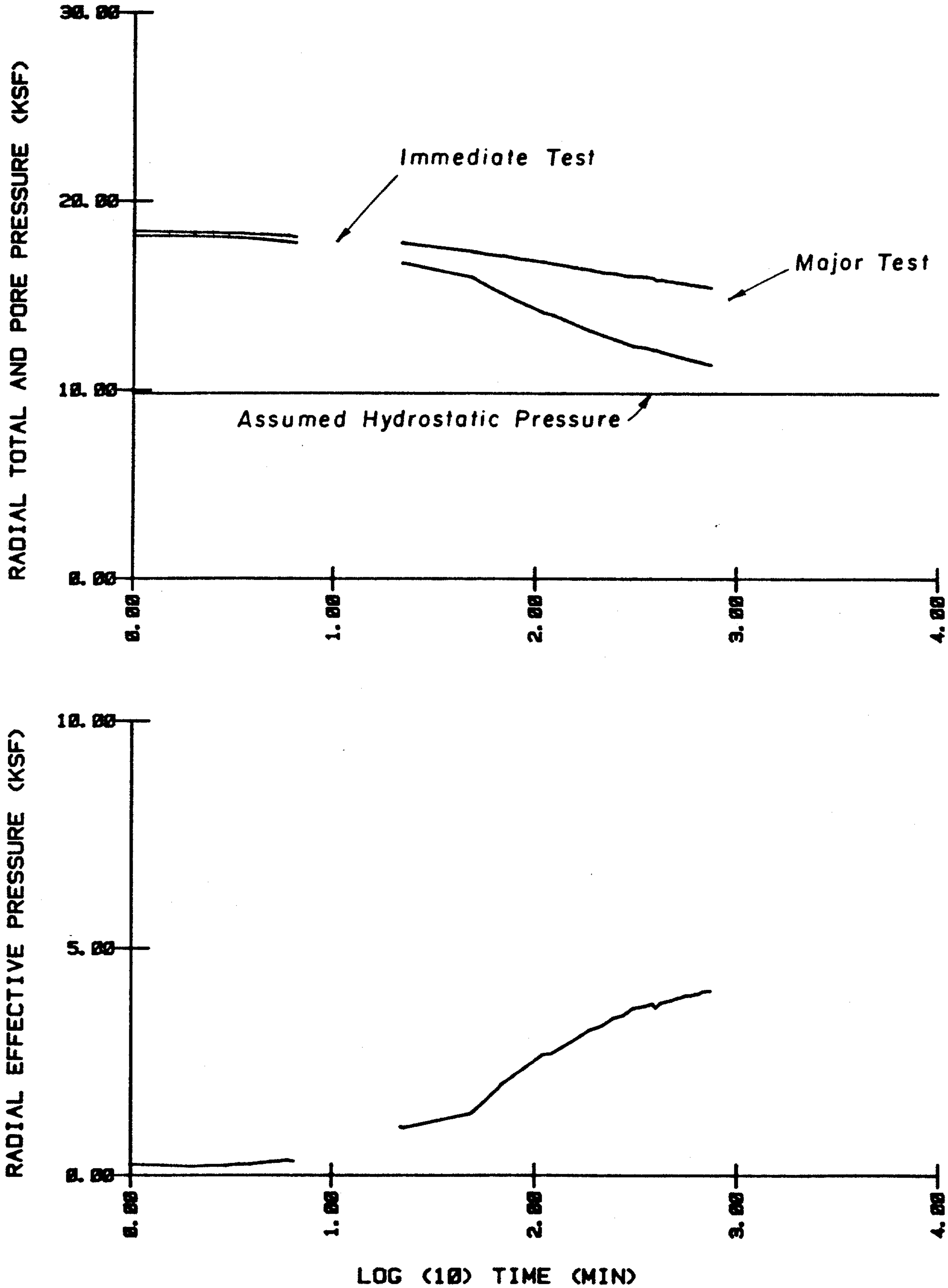
The load rate was then reduced to 0.0005 in./sec, with the results shown in Plate F-6. In this test, the peak shear transfer was 0.89 ksf, with a residual shear transfer value of 0.78 ksf.

The effects of load rate during this experiment were thus 22 percent per log cycle of increase in the slip rate.

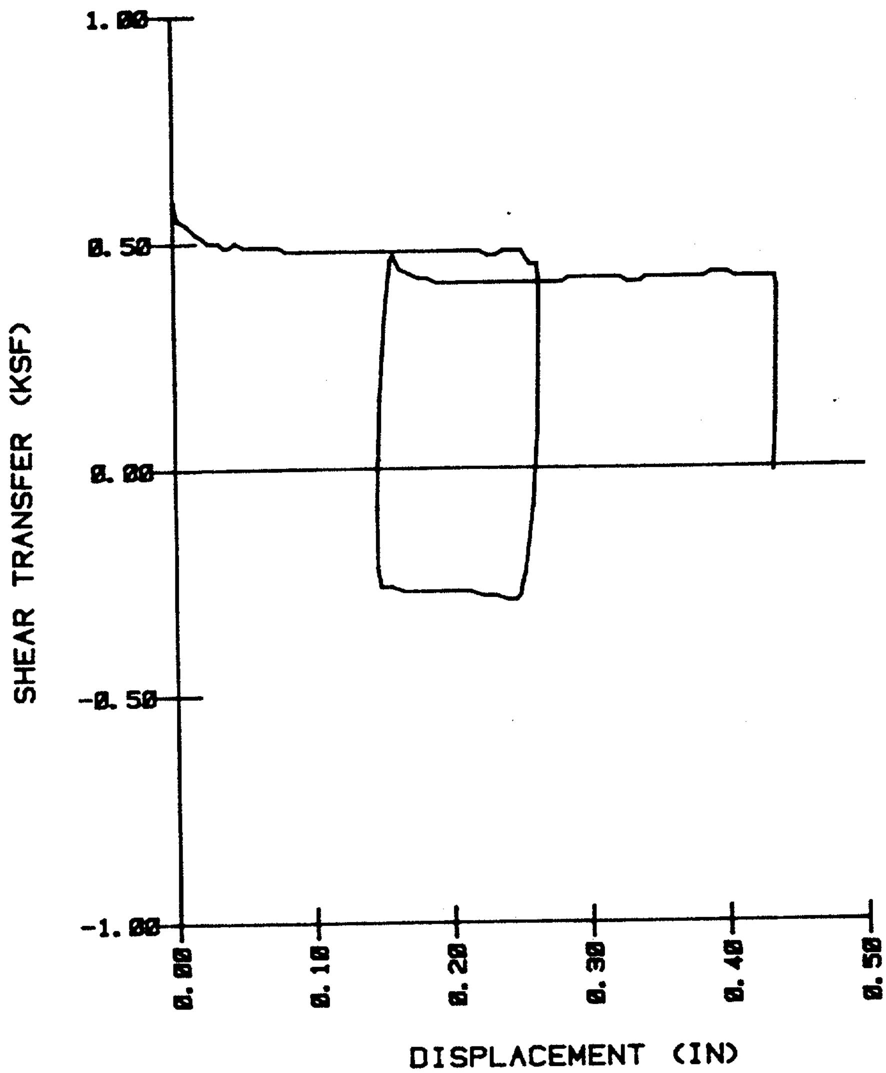
The fluctuations in the soil pressures during the major test series are given in Plate F-7. The abrupt increases in radial effective pressure, and the corresponding decreases in pore pressure, again coincide with the plastic-slip portion of the shear-displacement response.

The soil was then allowed to continued to consolidate for 4 1/2 hours. During this period, the total pressure slightly decreased from 14.7 ksf at the end of the cyclic tests to 14.5 ksf; the pore pressure decreased to 10.8 ksf, yielding net increase in the radial effective pressure from 3.4 to 3.7 ksf.

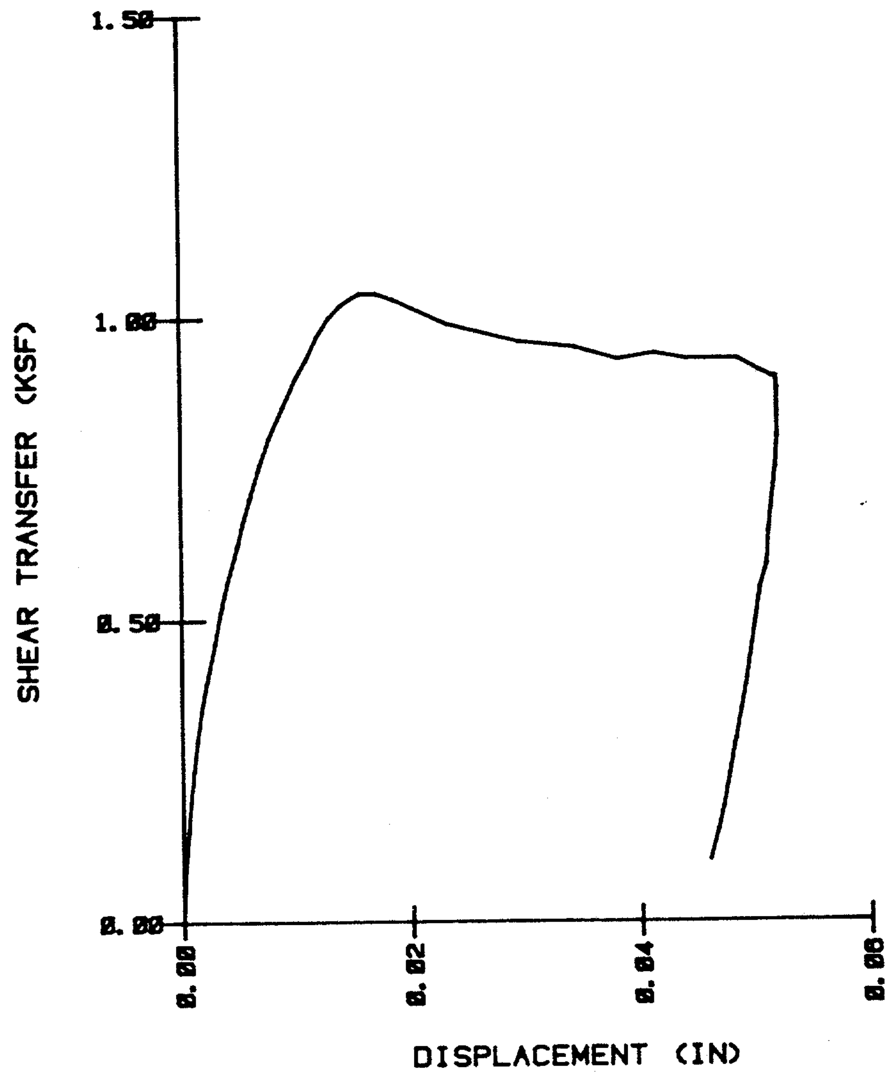
The probe was again loaded to failure, with the results shown in Plate F-8. The maximum shear transfer on the first loading was 0.98 ksf, with a residual shear value of 0.87 ksf. After the reversal of loading, the second tension peak was 0.95 ksf, with the residual shear transfer varying due to changes in the rate of displacement, which were higher than the initial loading. The loading shown in the plate was continued until the cutting shoe was pulled upward approximately 5 in. The probe was then removed.



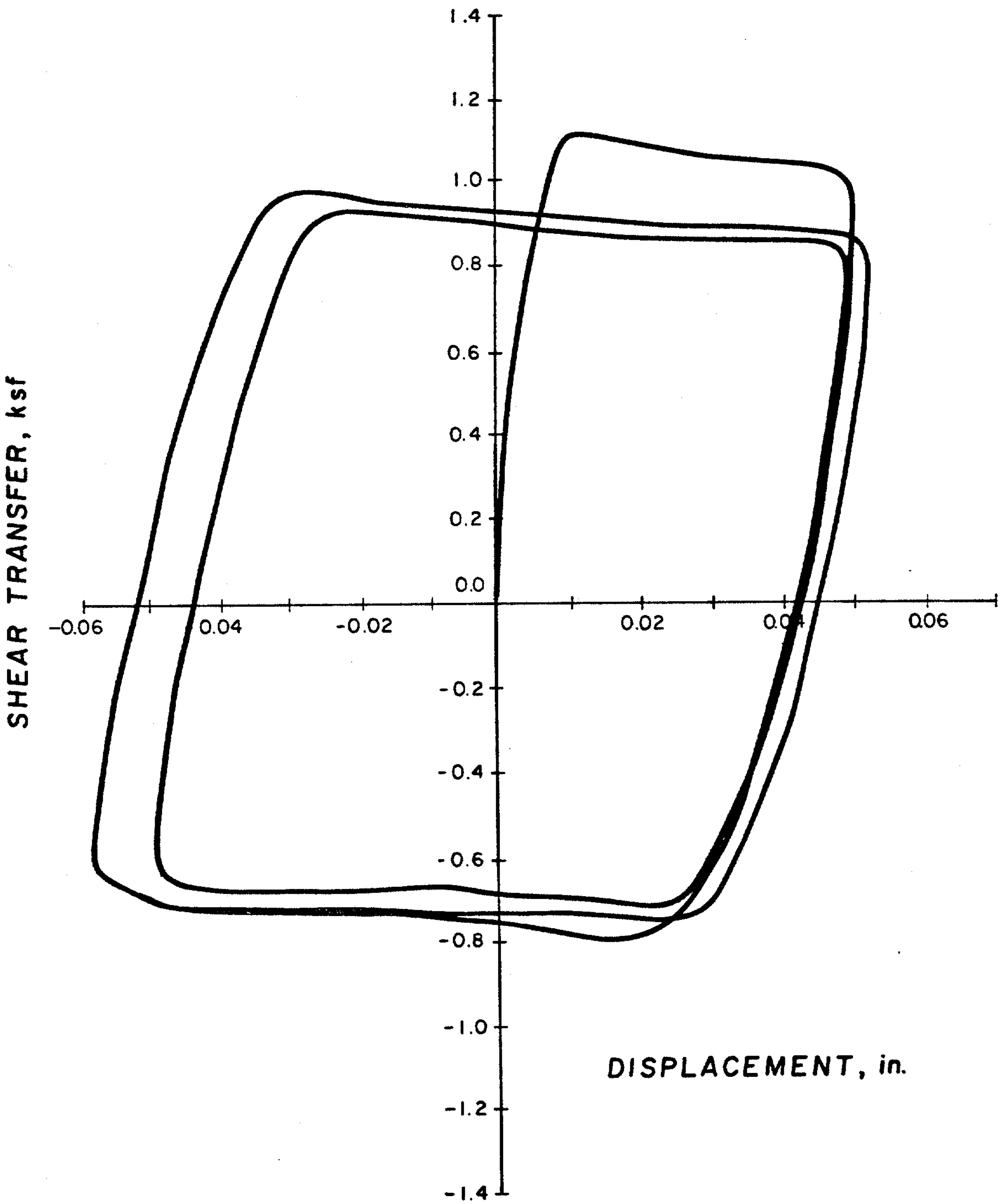
VARIATION IN SOIL PRESSURE DURING CONSOLIDATION



RESULTS OF THE LOAD TESTS IMMEDIATELY AFTER DRIVING



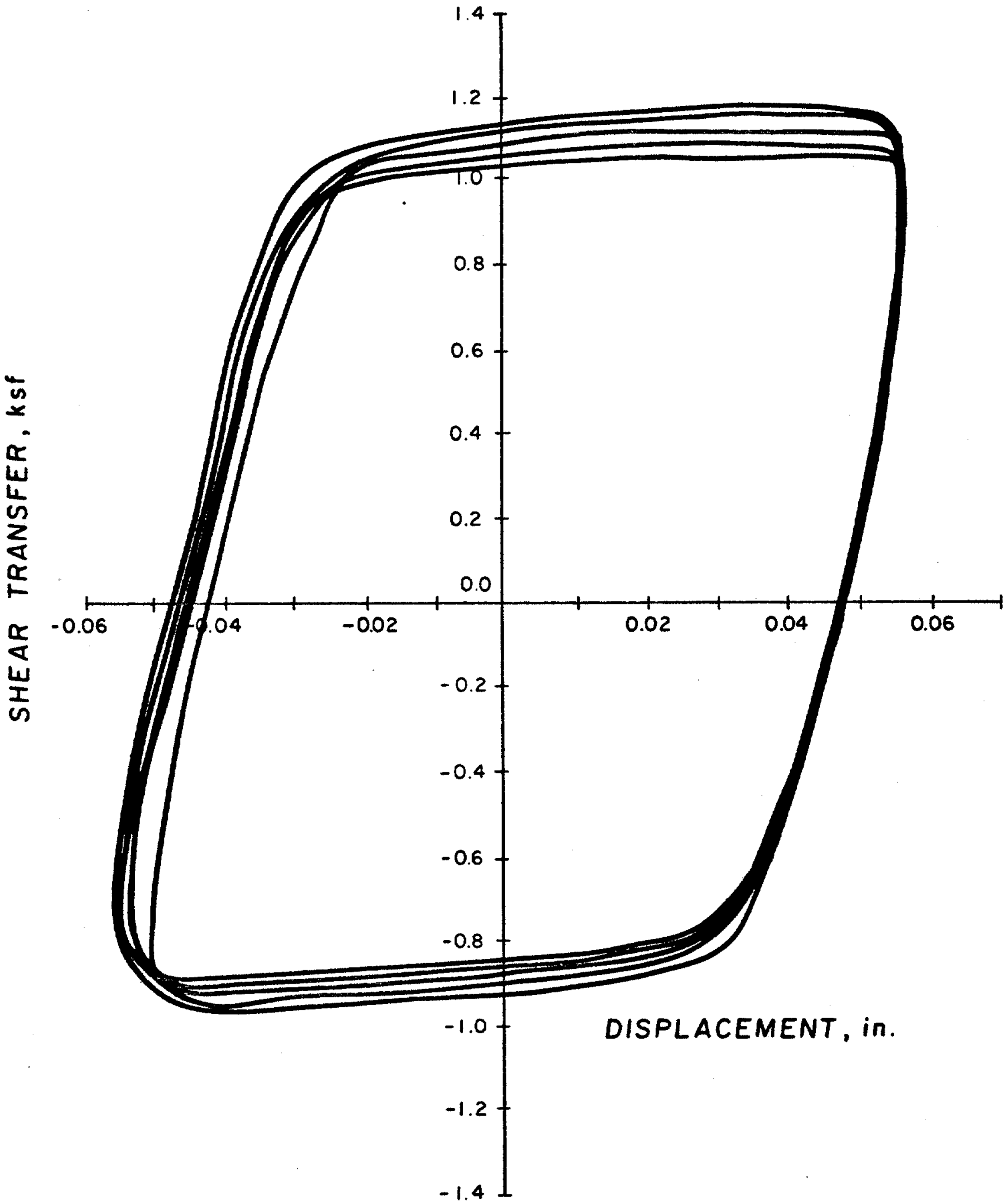
RESULTS OF THE INITIAL LOAD TEST AFTER CONSOLIDATION



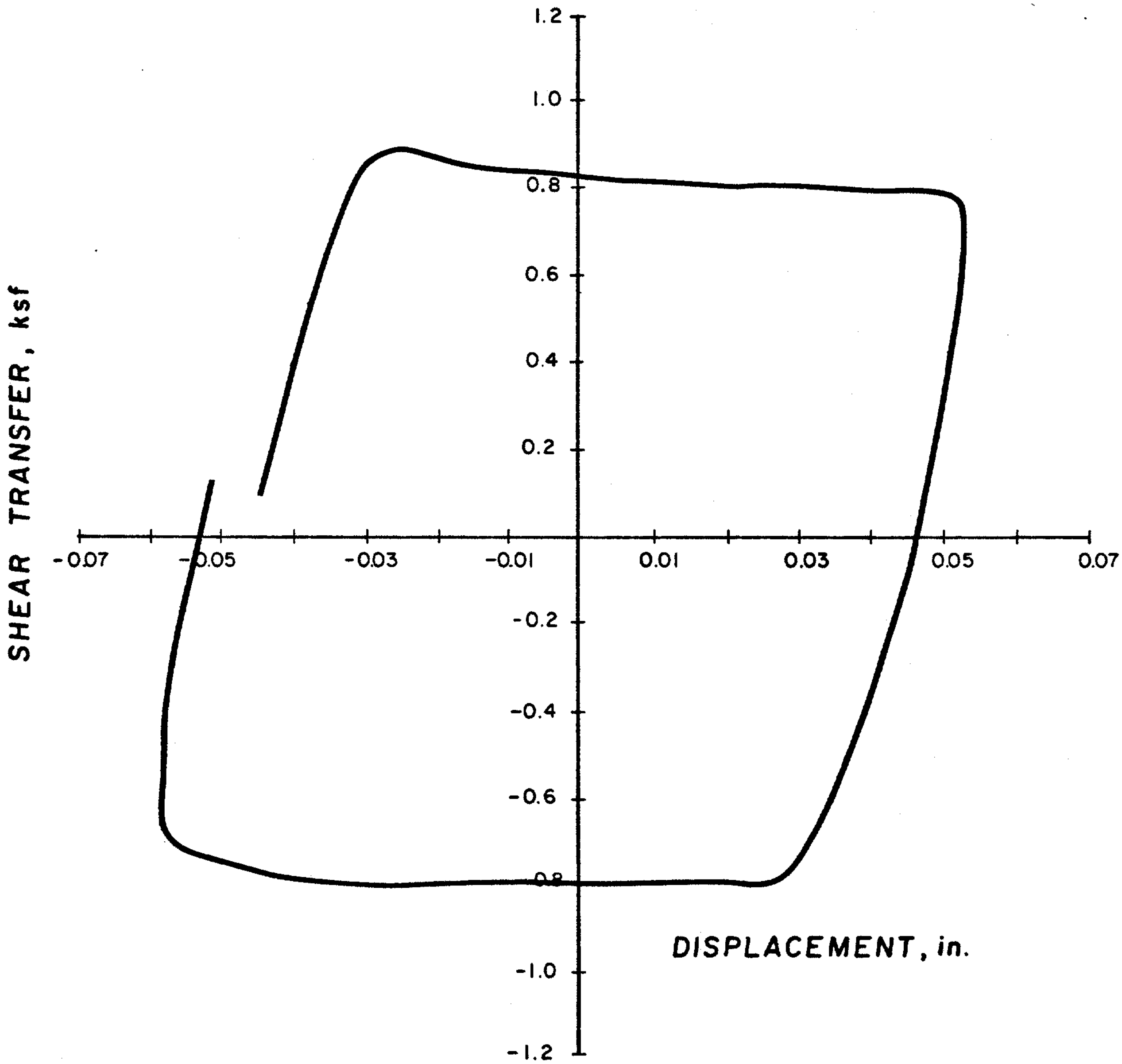
RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST

JOB NO.

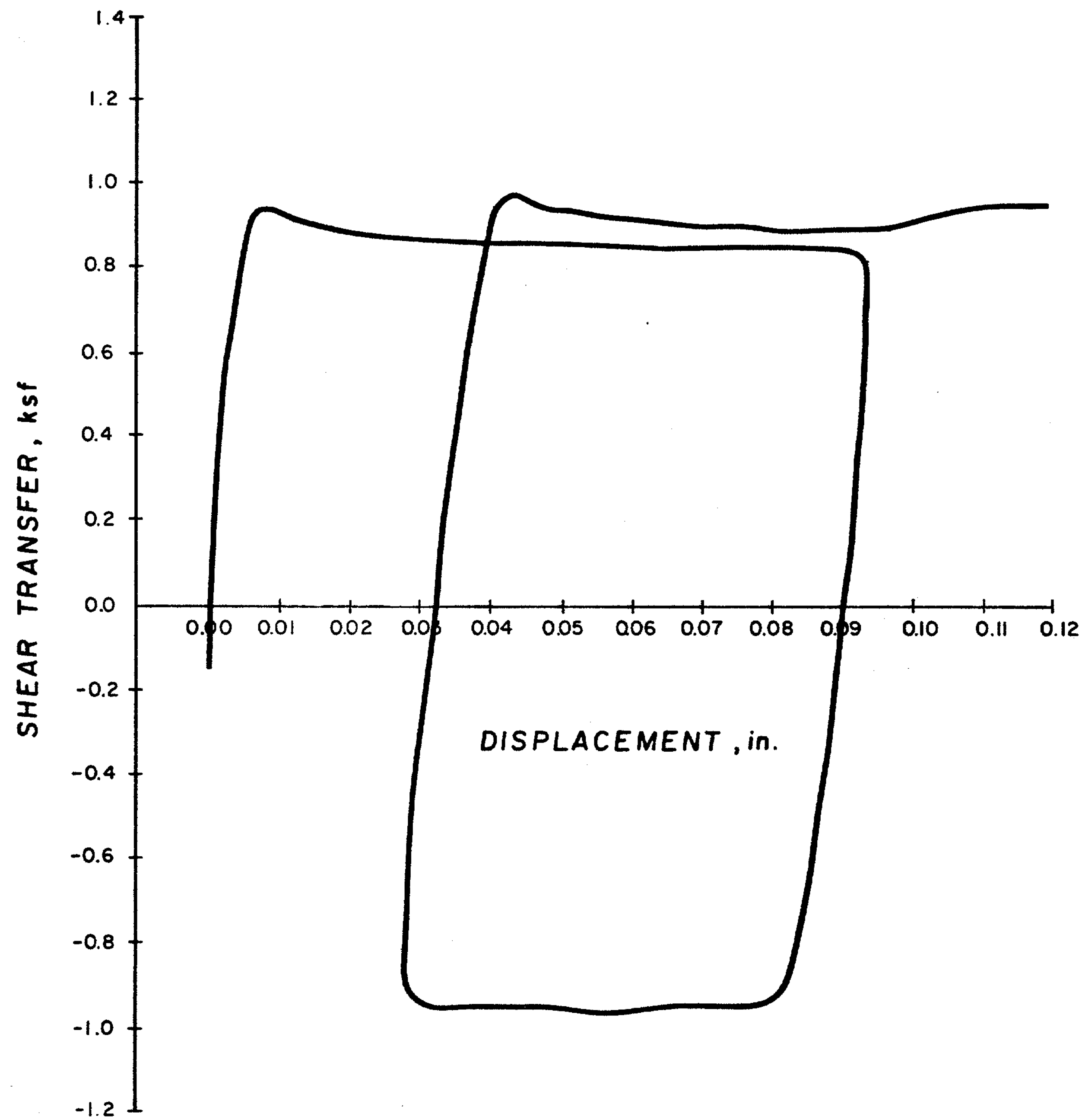
JOB NO. ______



RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST



RESULTS OF THE REPEAT OF THE TWO-WAY CYCLIC TEST



RESULTS OF THE LOAD TEST AFTER ADDITIONAL CONSOLIDATION

APPENDIX G: X-PROBE EXPERIMENT AT THE 160-FT DEPTH

RESULTS OF THE X-PROBE EXPERIMENT AT THE 160-FT DEPTH

The X-probe experiment at the 160-ft depth was performed during the period from 1100 hours on 17 April until 0800 hours on 19 April.

The probe was pushed into the soil using the drawdown on the drilling rig, with the push ending at 11:12 on 17 April. The variation in soil pressures following the push are given in Plate G-1. The maximum total pressure recorded after insertion was 28.7 ksf; the maximum pore pressure was 26.5 ksf.

The first load test was performed 15 minutes after insertion, at which time the total radial pressure was 27.1 ksf and the pore pressure was 24.3 ksf, for a radial effective pressure of 2.8 ksf. The results of the load test are given in Plate G-2, and indicated a peak shear transfer on the first loading of 0.37 ksf, with a residual shear transfer of 0.34 ksf. The residual shear transfer after reversal of loading increased, to 0.36 ksf.

Consolidation was allowed to proceed for almost 24 hours, during which time the total radial pressure had decreased to 21.1 ksf, the pore pressure decreased to 13.5 ksf, yielding an increase in the radial effective pressure from 3.4 ksf to 7.6 ksf.

The probe was then loaded to failure in tension, with the results being given in Plate G-3. The maximum shear transfer during this loading was 1.18 ksf.

The pore pressures were then monitored for 80 minutes, during which time the total radial pressure decreased slightly from 20.9 ksf to 20.8 ksf, the pore pressure decreased from 14.7 ksf to 13.9 ksf, yielding an increase in the radial effective pressure from 6.2 ksf to 6.9 ksf.

The probe was then subjected to a sequence of cyclic (repeated) tension tests, with a summary of the results shown in Plate G-4. As shown in the plate, the accumulation of permanent upward displacement began at load levels greater than 60 percent of the maximum resistance in the test to failure. An increase in the load level to 89 percent of the initial static capacity increased the rate of accumulation of displacement, but the relationship was still linear after 30 cycles. The absence of any tendency for the rate of accumulation of permanent displacement to increase during these 30 cycles again suggested that no degradation in resistance was occuring.

The fluctuations in the soil pressures during the one-way cyclic tension tests are shown in Plate G-5. No general trends can be seen in either the pore pressure or the radial effective pressure, for either an increase or decrease in the mean value.

The probe was then loaded to failure in tension, with the results shown in Plate G-6. The shear transfer at yield was 1.09 ksf; the shear transfer just prior to the load reversal was 1.18 ksf.

The probe was then subjected to six cycles of two-way controlled-displacement loading, with the results given in Plate G-7. The maximum shear transfer on the first cycle was 1.15 ksf. On the sixth cycle, the peak shear transfer was 1.12 ksf, with a residual shear transfer of 0.96 ksf.

The loading rate was then increased, with the results shown in Plate G-8. The peak shear transfer at yield on the fifth cycle was 1.06 ksf; the residual shear was 1.00 ksf. During these tests, the rate of plastic slip was 0.0149 in./sec.

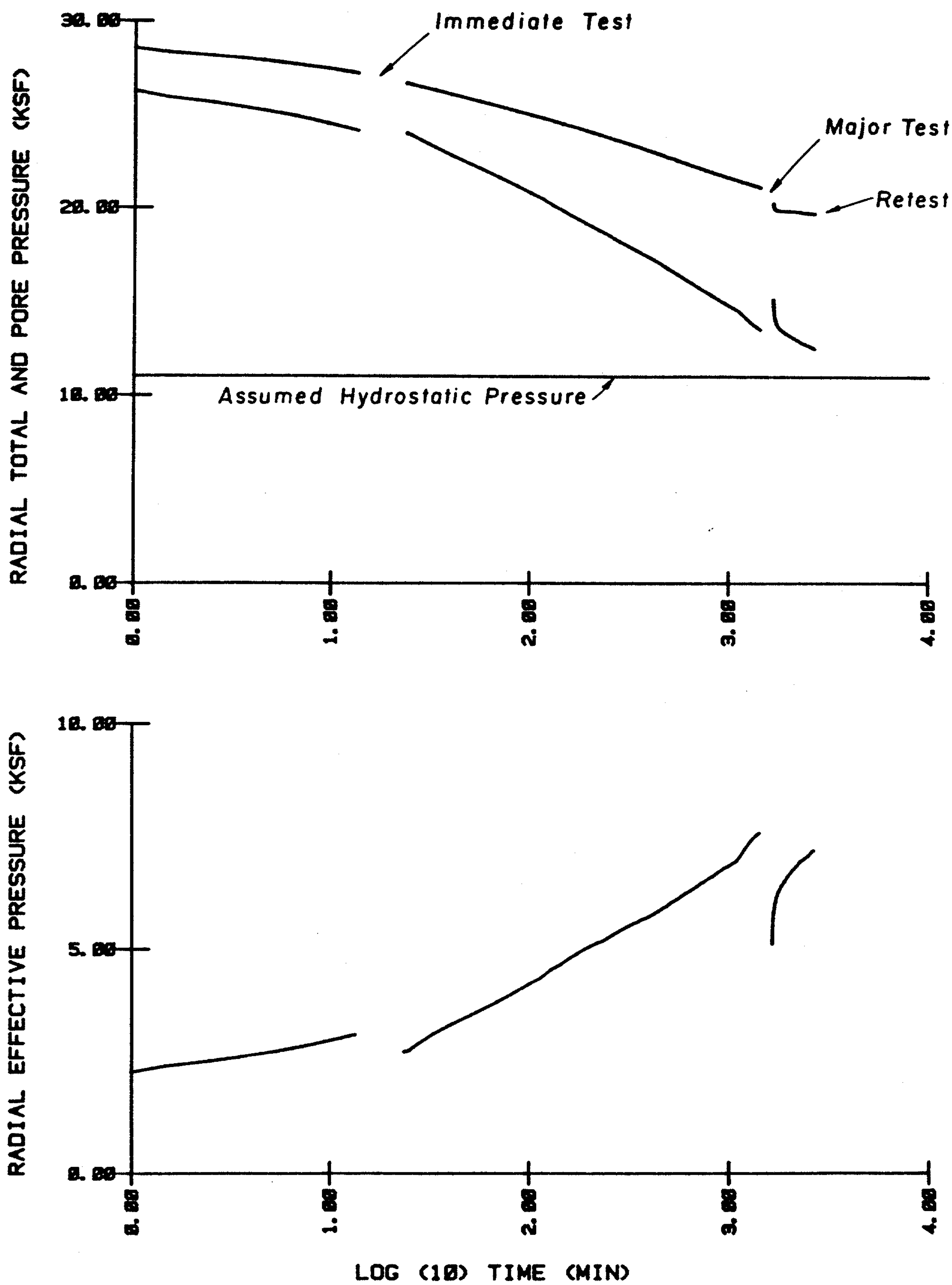
The load rate was then decreased to the slow rate, with the results shown in Plate G-9. The peak shear transfer at yield was 1.11 ksf; the residual shear transfer was 0.93 ksf, at a slip rate of 0.00056 in./sec.

Again, the loading rate has not affected the value of shear transfer at yield, but the plastic shear was increased by about 5 percent per log cycle of increase in slip rate.

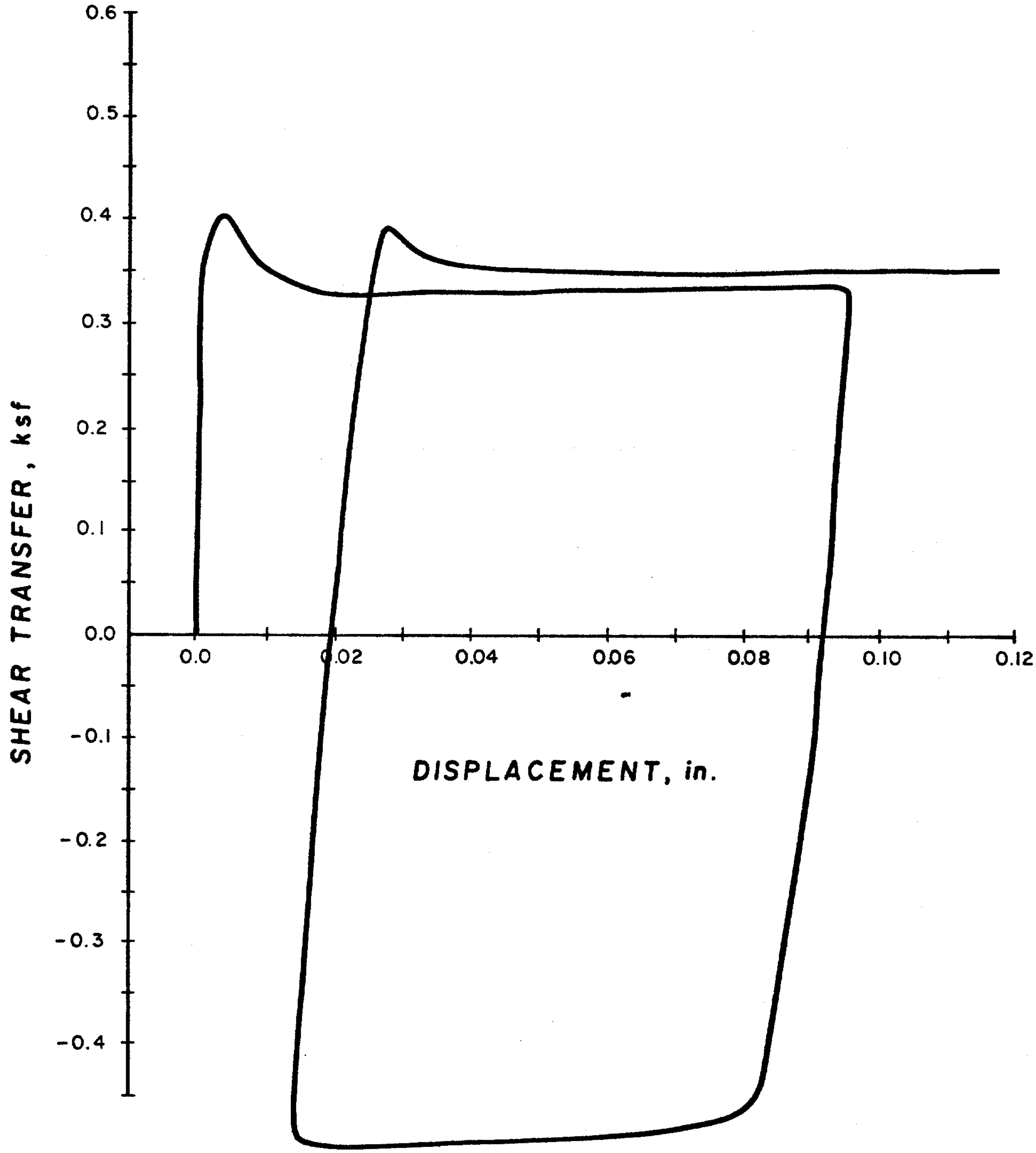
The fluctuations in soil pressure during the two-way cyclic tests are shown in Plate G-10. Again, the increases in effective pressure occur during plastic slip.

Consolidation was allowed to continue for 17 hours, during which time the radial total pressure decreased from 20.3 ksf to 19.7 ksf, the pore pressure decreased from 15.1 ksf to 12.5 ksf, and the radial effective pressure increased from 5.2 ksf to 7.2 ksf.

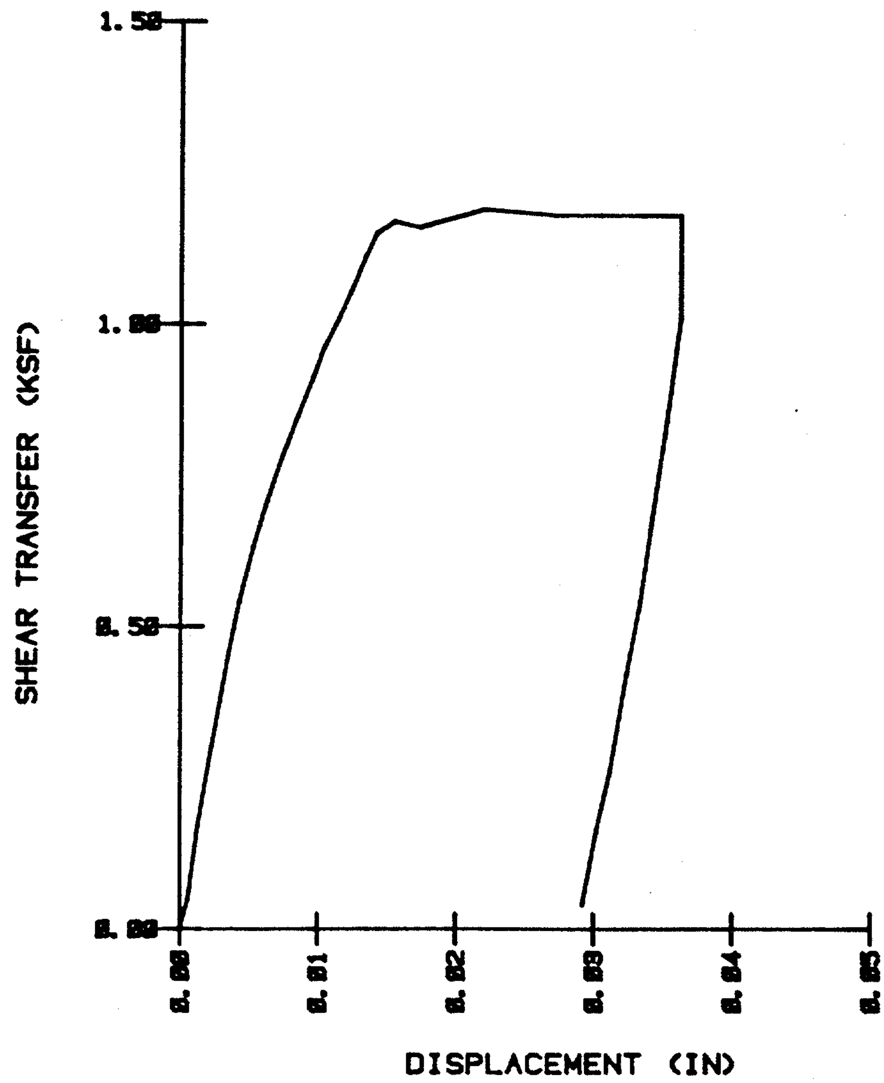
The probe was then loaded to failure in both tension and compression, as shown on Plate G-11. The peak shear transfer on the first loading was 1.41 ksf; the residual shear transfer was 1.15 ksf. This value of shear transfer agrees very well with the value of 1.42 ksf for the estimated undrained shear strength at this depth.



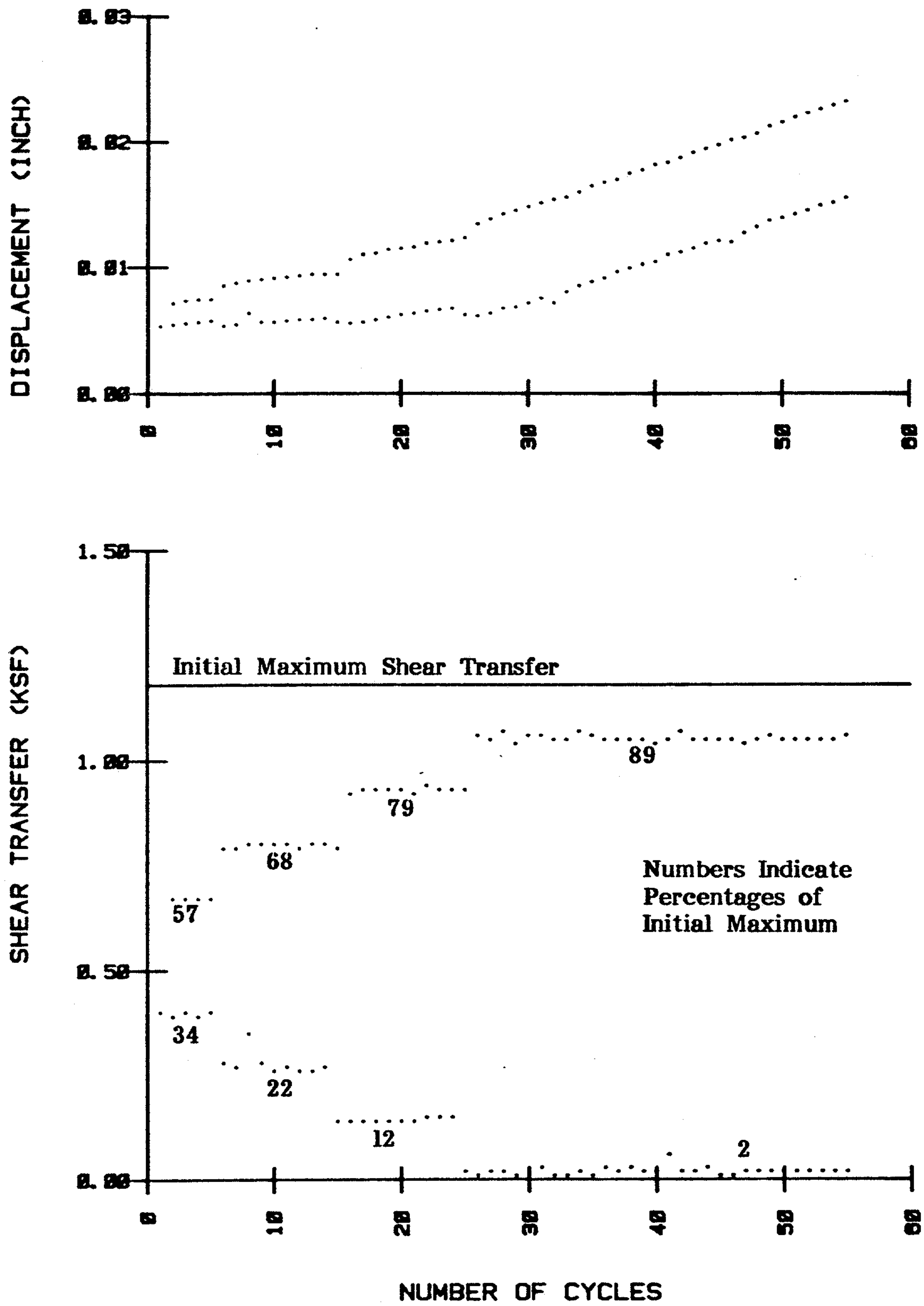
VARIATION IN SOIL PRESSURE DURING CONSOLIDATION



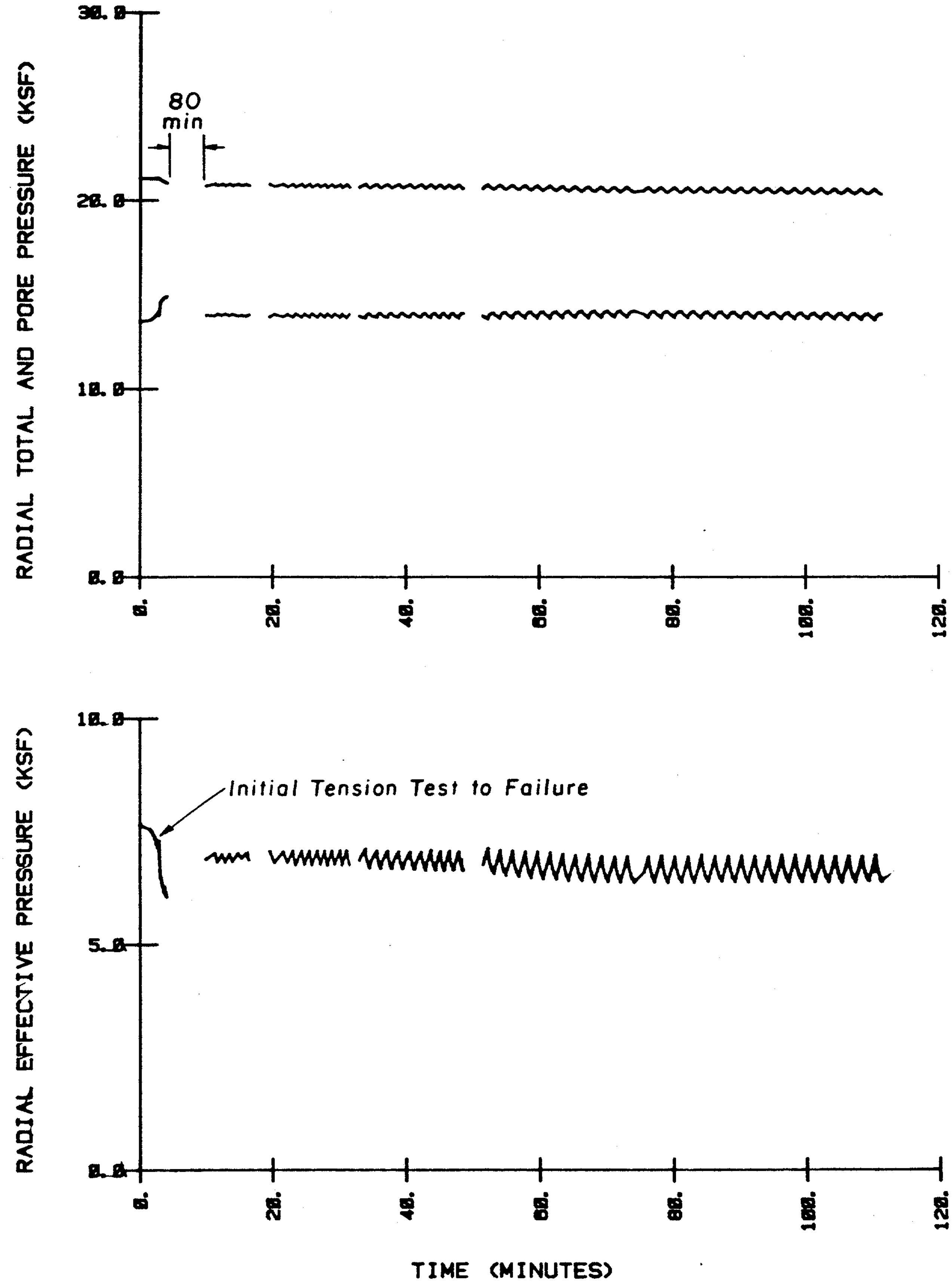
RESULTS OF THE LOAD TESTS IMMEDIATELY AFTER INSTALLATION



RESULTS OF THE INITIAL TEST TO FAILURE AFTER CONSOLIDATION



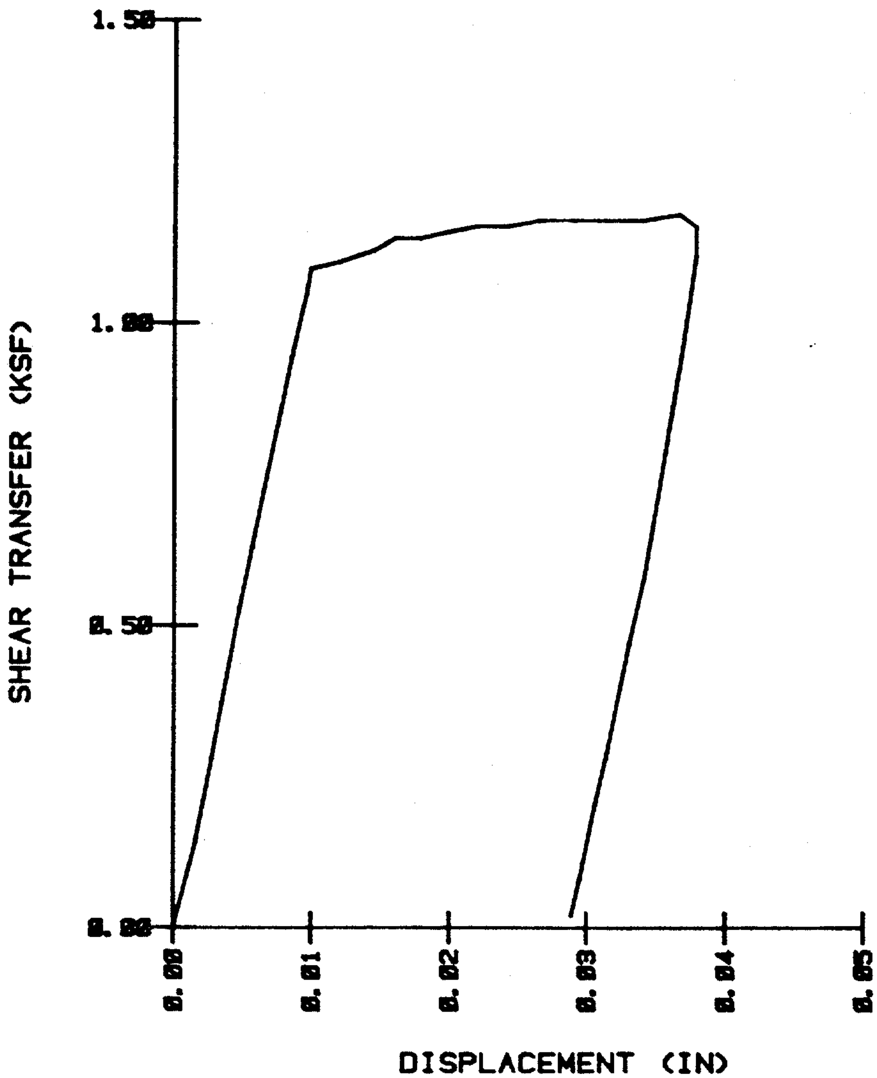
RESULTS OF THE ONE-WAY CYCLIC TENSION TESTS



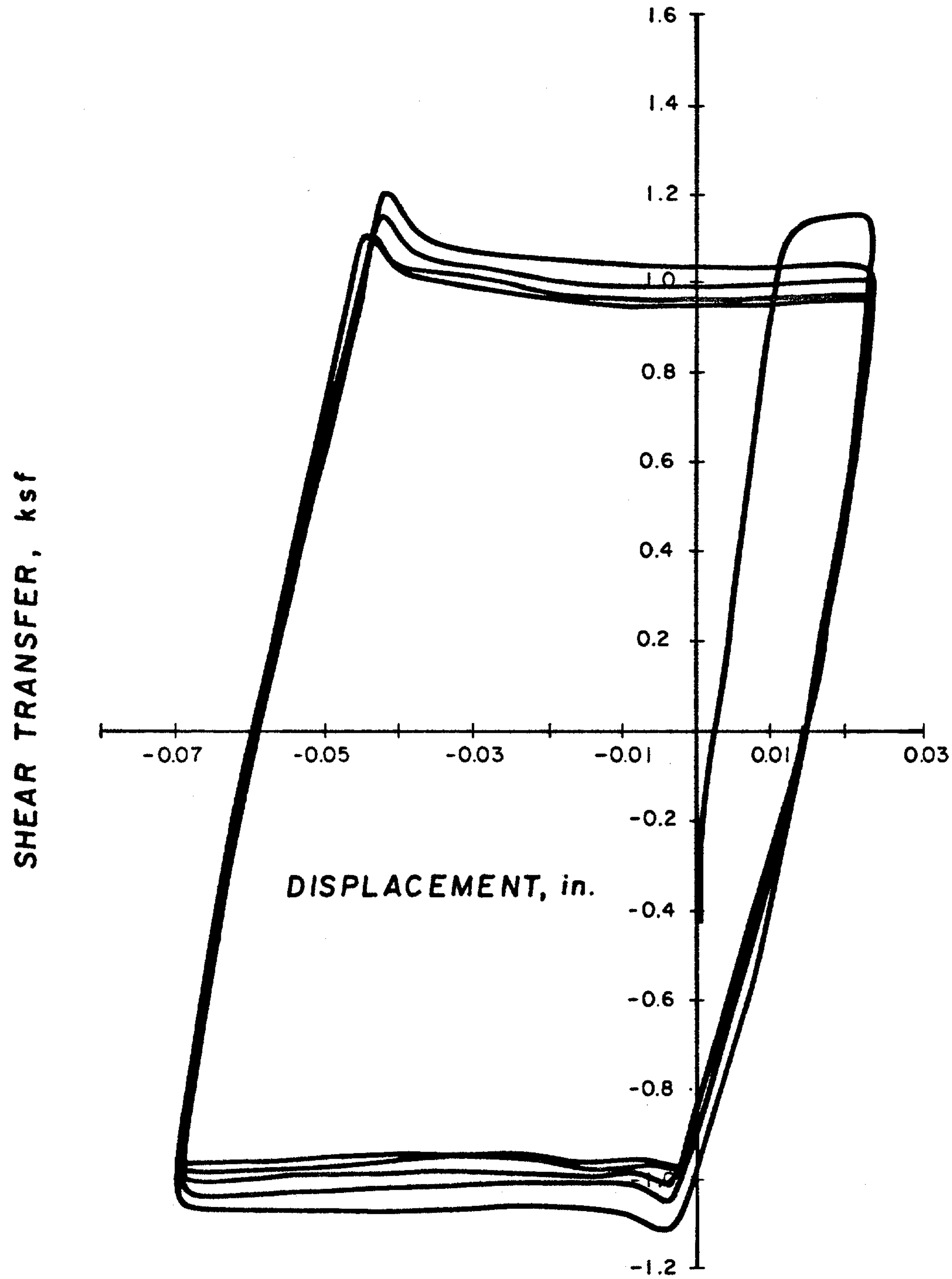
FLUCTUATIONS IN SOIL PRESSURE DURING ONE-WAY TENSION TESTS

JOB NO.

The Earth Technology Corporation

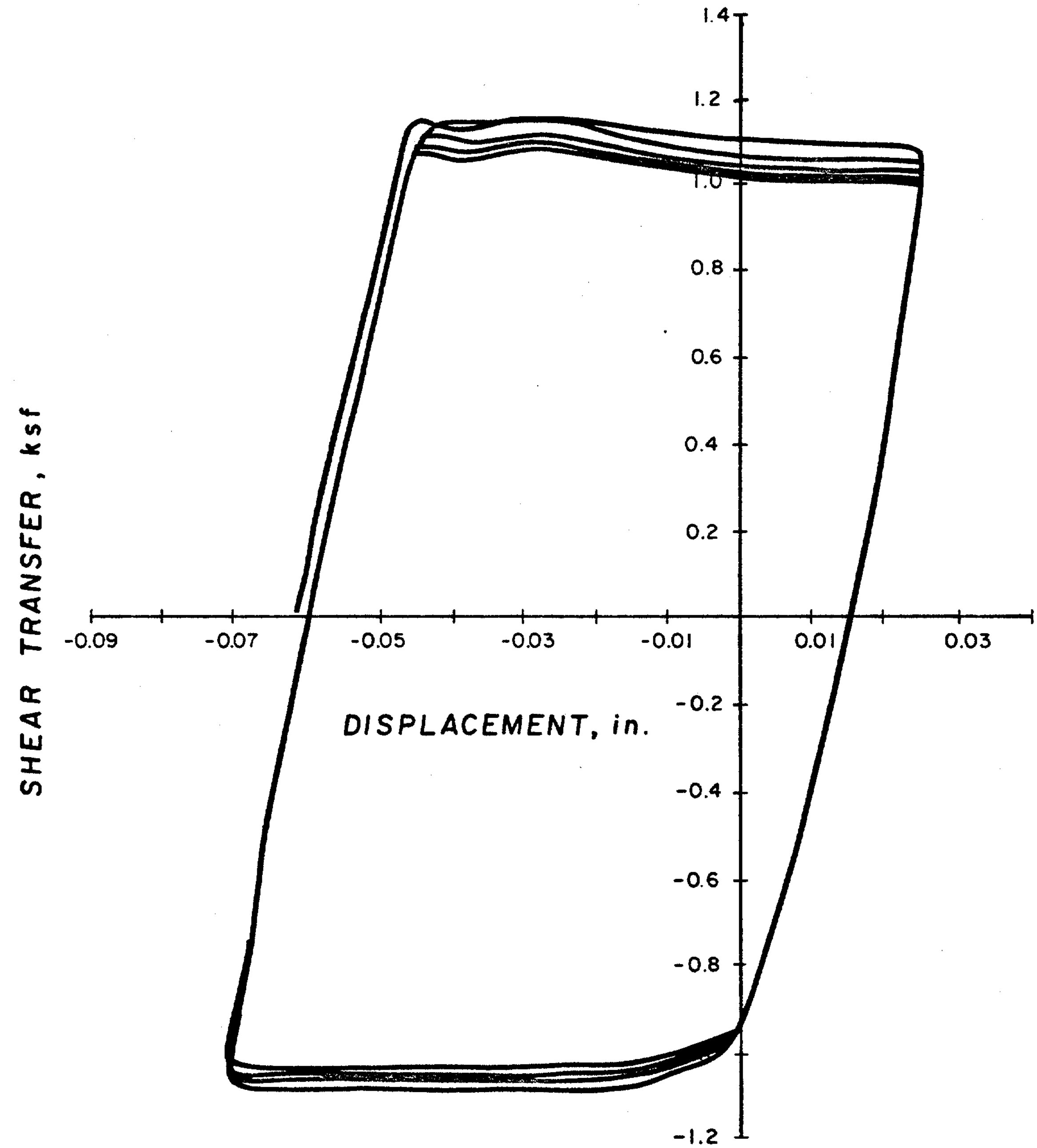


RESULTS OF THE LOADING TO FAILURE AFTER THE ONE-WAY CYCLIC TENSION TESTS

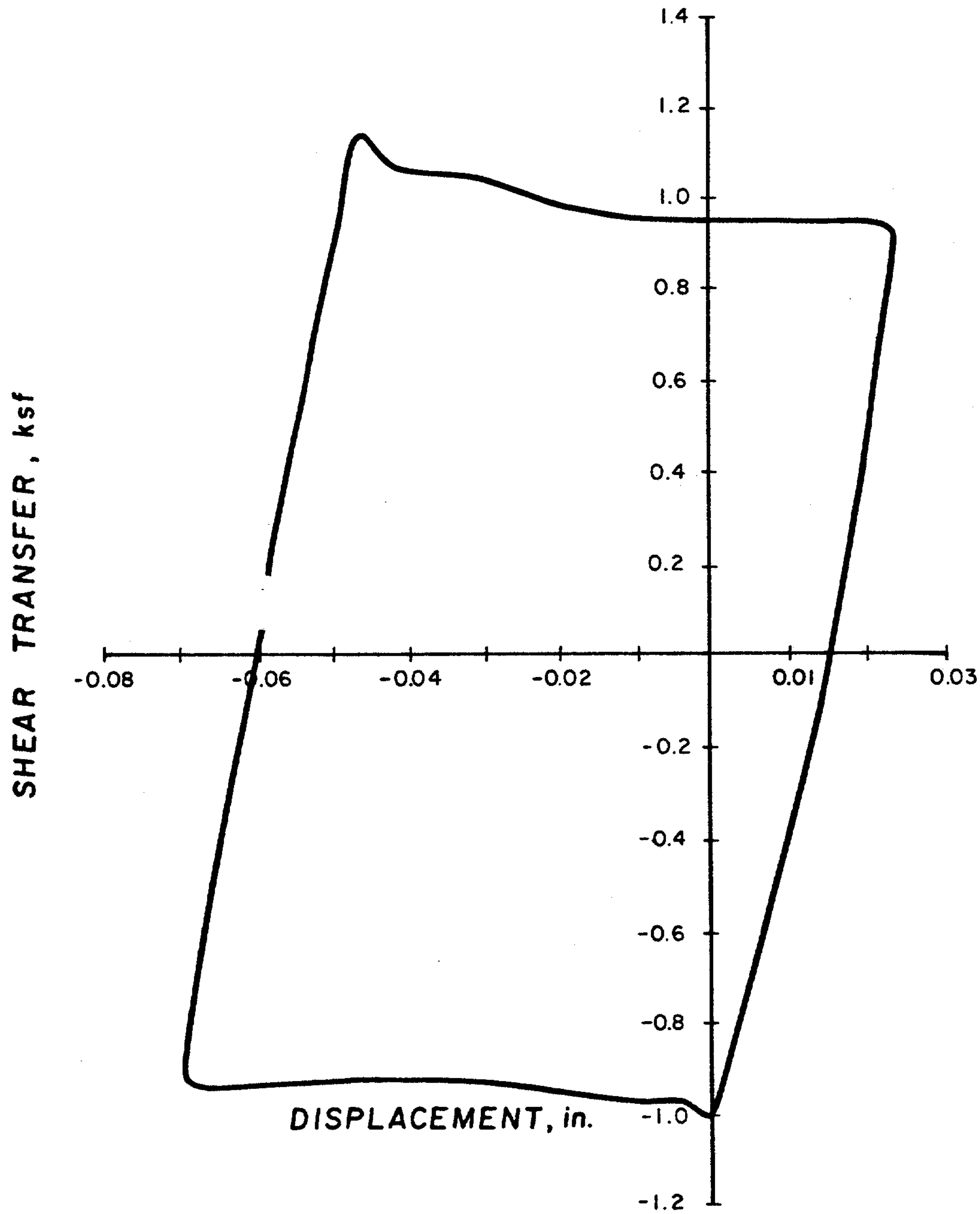


RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST

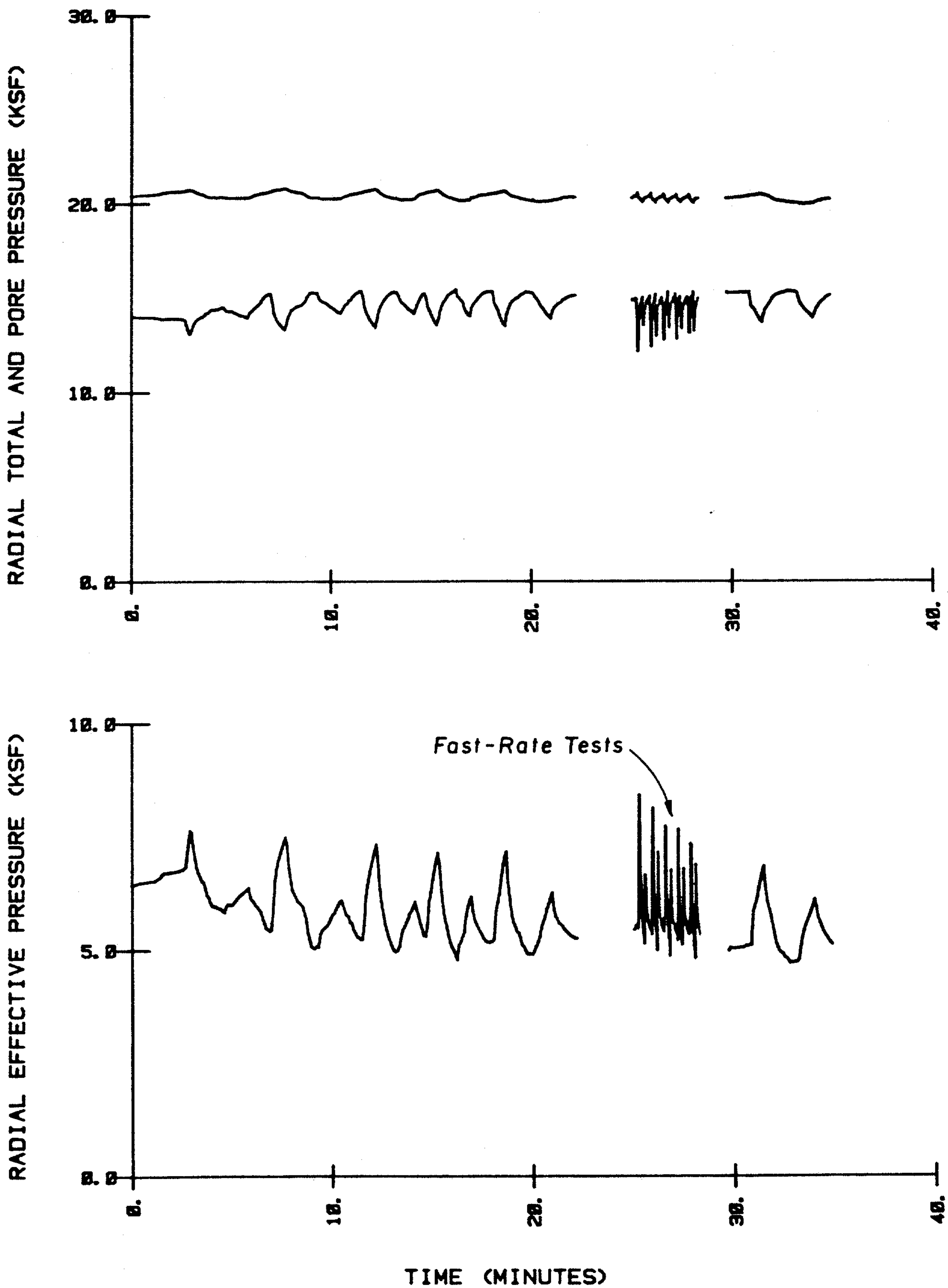
JOB NO



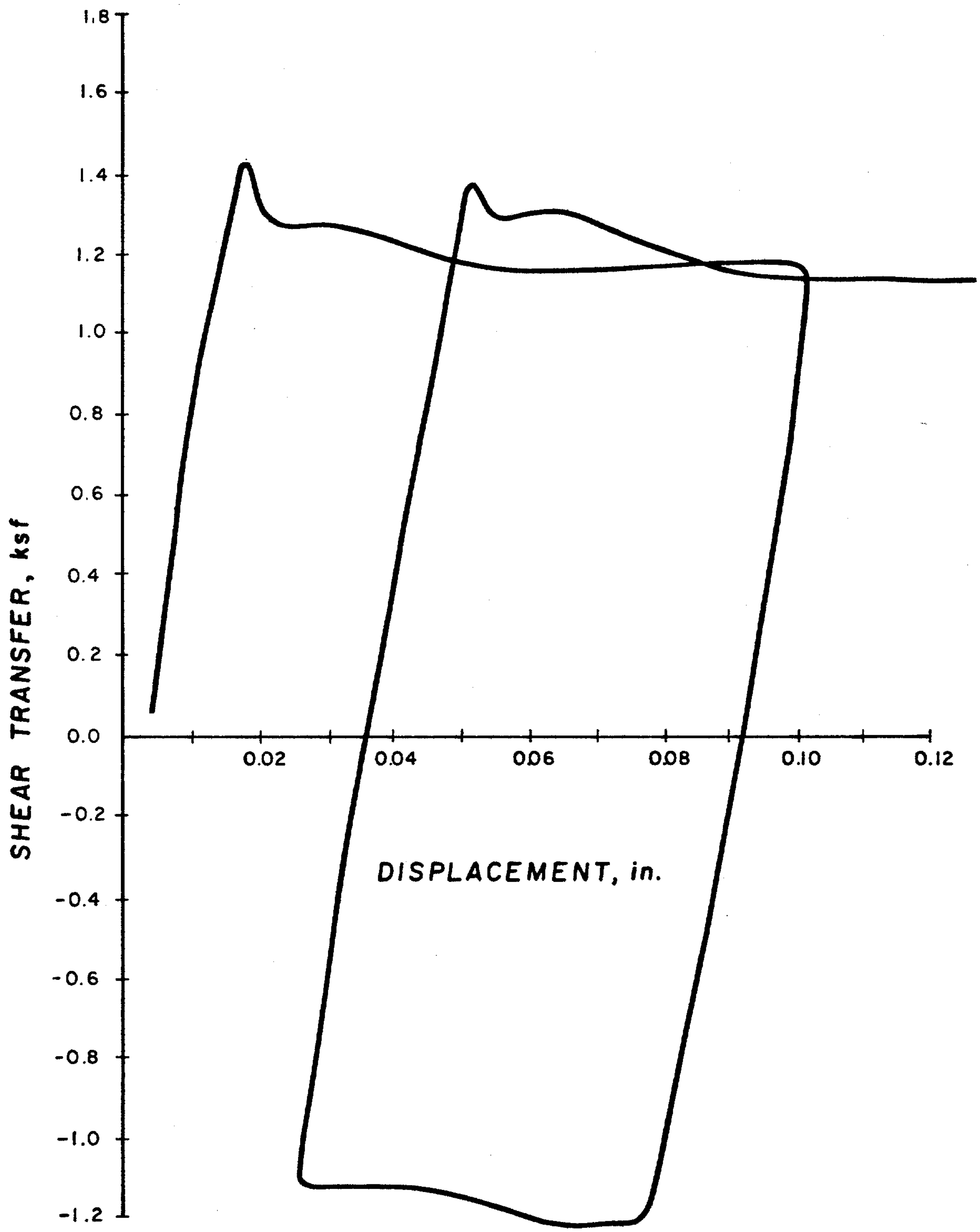
RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST



RESULTS OF THE REPEAT OF THE TWO-WAY CYCLIC TEST



FLUCTUATIONS IN SOIL PRESSURE DURING TWO-WAY CYCLIC LOADING



RESULTS OF THE LOAD TEST AFTER ADDITIONAL CONSOLIDATION

APPENDIX H: CLOSED-END 3-IN. PROBE EXPERIMENT AT THE 160-FT DEPTH

RESULTS OF THE CLOSED-END 3-IN. PROBE EXPERIMENT AT THE 160-FT DEPTH

The experiment with the closed-end 3-in. probe was performed during the period from 1430 hours on 15 April until 0900 hours on 19 April.

The probe was installed by driving, with the process begun at 14:45 and ending at 14:59.

The variation in soil pressures after installation are given in Plate H-1. As seen in the plate, the decreases in both the total and pore pressures during consolidation result in a linear increase in the radial effective presssure, when plotted against the logarithm of time.

The first load test was performed 25 minutes after driving. During this 25-minute period, the total pressure decreased from a maximum value of 29.4 ksf to 28.6 ksf, and the pore pressure decreased from a maximum value of 26.9 ksf to 25.4 ksf, yielding an increase in the radial effective pressure from 2.5 ksf to 3.2 ksf.

The results of the first load test are given in Plate H-2, and show a maximum shear transfer of 0.50 ksf, with the value decreasing to 0.25 ksf after the reversal of direction. The wavy nature of the curve is due to variations in load rate; one of the solenoid valves had an intermittent electrical short, and was not functioning properly. After this test, the valve was replaced.

As shown in Plate H-1, the consolidation was allowed to proceed for 73 hours. During this period, the total pressure decreased from 27.7 ksf to 24.1 ksf, and the pore pressure decreased from 25.2 ksf to 13.8 ksf, yielding an increase in the radial effective pressure from 2.5 ksf to 10.3 ksf.

The probe was then loaded to failure in tension, with the results shown in Plate H-3. The peak shear transfer was 1.48 ksf, with a residual shear transfer of 1.22 ksf. This value agrees well with the estimated undrained shear strength at this depth of 1.42 ksf.

A period of 80 minutes was allowed for the pore pressures to recover. During this period, the total pressure changed only slightly, from 23.6 ksf to 23.7 ksf, the pore pressure decreased from 16.0 ksf to 14.4 ksf, with the radial effective pressure thus increasing from 7.6 ksf to 9.3 ksf.

A sequence of one-way cyclic (repeated) tension tests were then applied to the probe, with the results shown in Plate H-4. As in the earlier experiments, the probe began to accumulate permanent upward displacements at load levels greater than 60 percent of the initial maximum shear transfer from the static test. Except for the sequence of loads to levels exceeding the initial maximum shear transfer, the accumulation of displacement is again linear, suggesting that the behavior is simply that of an inelastic system, without any cyclic degradation.

The fluctuations in the soil pressures during this series of tests are shown in Plate H-5. As seen in the plate, there is a trend for the radial effective pressure to slightly decrease during the tests, with the rate of loss increasing when the maximum tension loads equal that measured during the initial test to failure.

The probe was then subjected to a slow monotonic loading to failure in tension followed by five cycles of displacement-controlled two-way cycling. The results of these are shown in Plate H-6. The maximum shear transfer onthe first loading was 1.49 ksf; after five cycles, the peak shear transfer was 1.07 ksf, with a residual shear transfer of 1.02 ksf.

The rate of loading was then increased, with the results shown in Plate H-7. On the sixth cycle, the limiting shear transfer was 1.11 ksf, at a rate of plastic slip of 0.0138 in./sec.

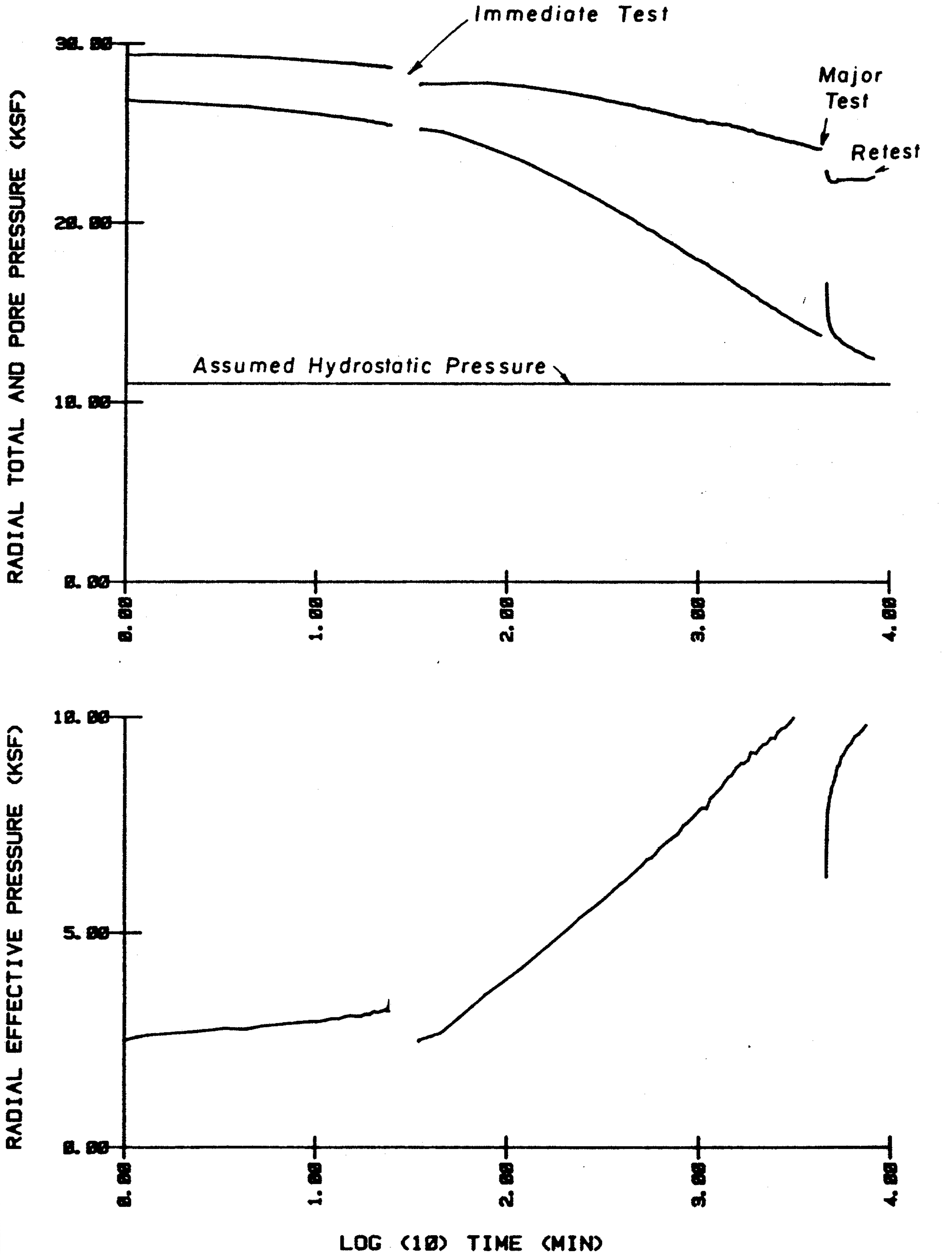
The loading rate was then reduced, with the results shown in Plate H-8. The peak shear on the second cycle was 0.94 ksf, with a residual shear transfer of 0.89 ksf.

The rate of slip on the slow test was 0.00095 in./sec, which indicates an increase of about 21 percent per log cycle of increase in slip rate.

The fluctuations in the soil pressures during the two-way cyclic tests are shown in Plate H-9. Again, the decreases in pore pressure and increases in the radial effective pressure occurred during constant-shear plastic slip.

As seen in Plate H-1, consolidation was then allowed to proceed for 71 additional hours. During this period, the total pressure remained at a reasonably-constant value of 22.5 ksf (after an initial fluctuation), and the pore pressure decreased from 15.3 ksf to 12.5 ksf, yielding an increase in the radial effective pressure from 7.1 ksf to 10.2 ksf.

The probe was then subjected to a slow monotonic loading to failure in both tension and compression, with the results shown in Plate H-10. The peak shear transfer on the first loading was 1.61 ksf, with a residual shear transfer of 1.43 ksf. The rate of plastic slip was increased during the test, and showed an increase in shear transfer from 1.43 ksf to 1.84 ksf with an increase in the slip rate from 0.00041 in./sec to 0.0296 in./sec. This increase corresponds to about 15 percent per log cycle of increase in slip rate.

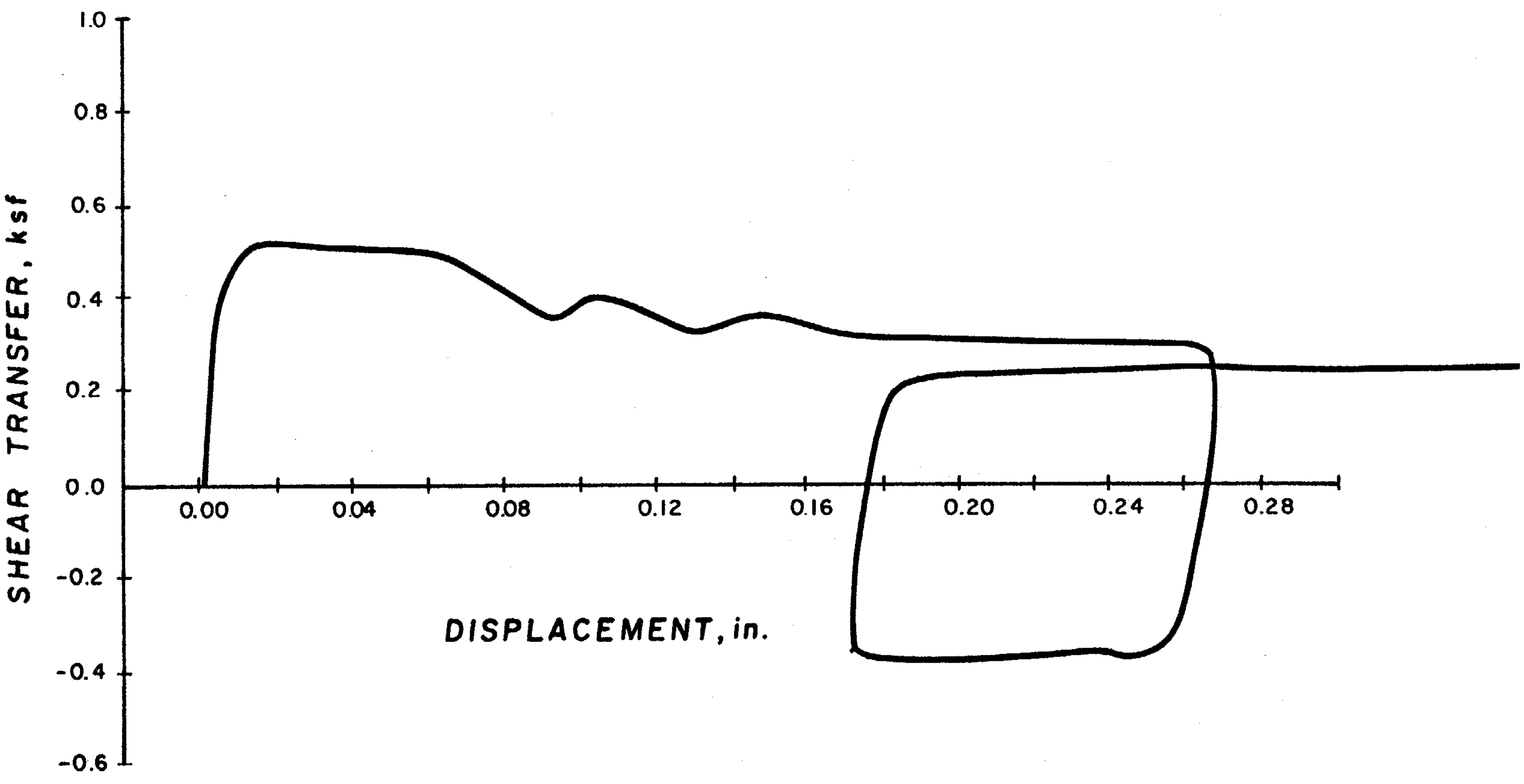


VARIATION IN SOIL PRESSURES DURING CONSOLIDATION

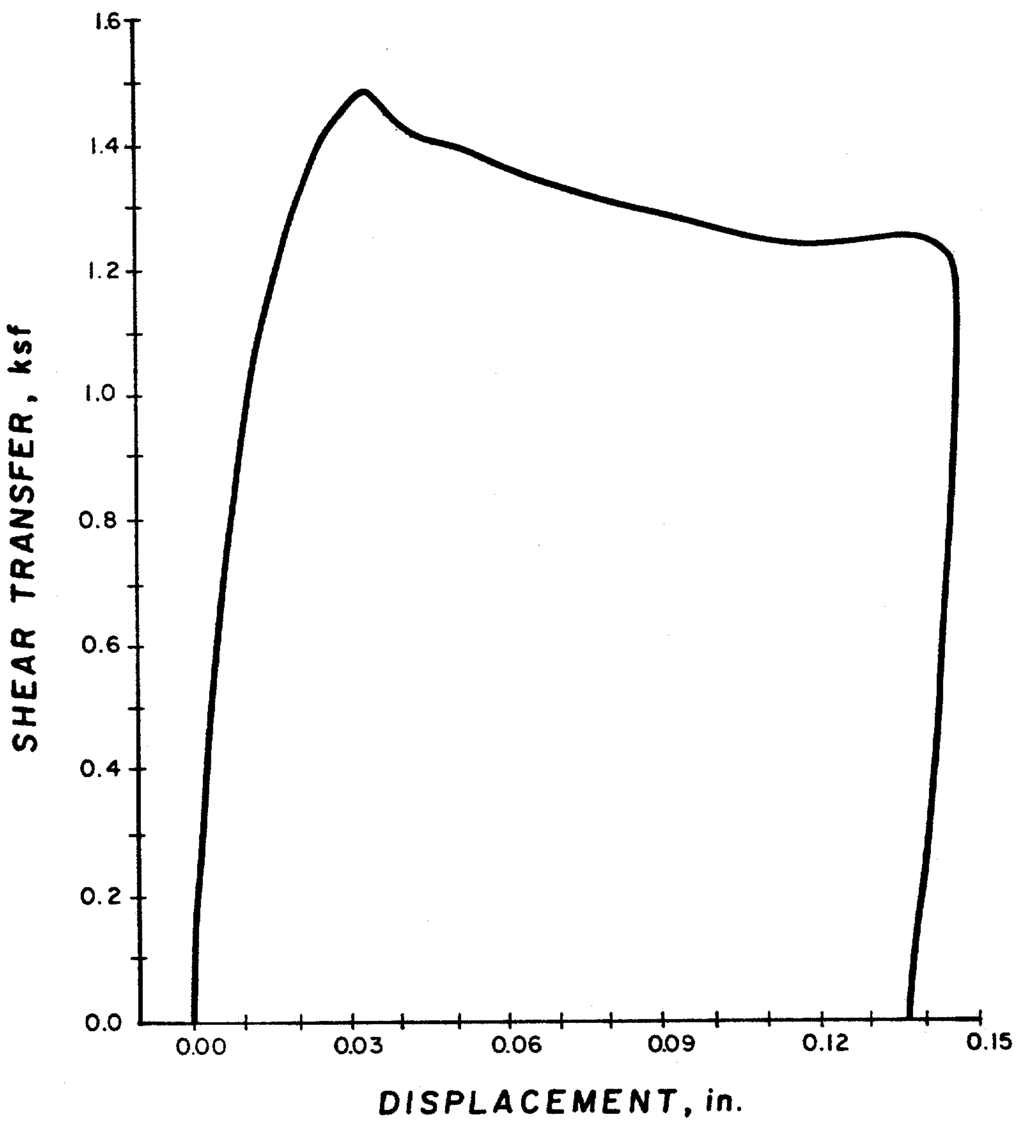
The Earth Technology Corporation

PLATE H-1

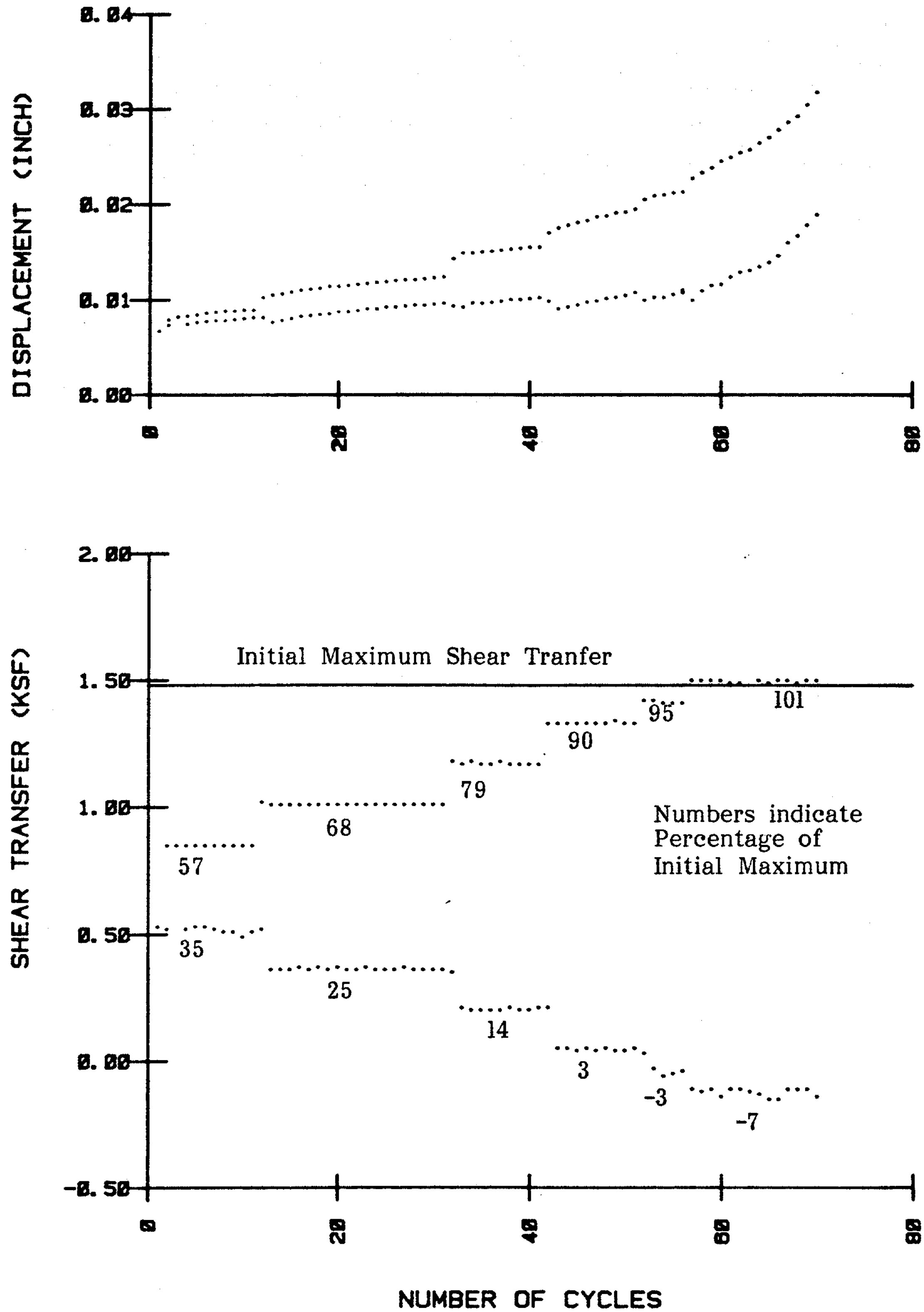
JOB NO.



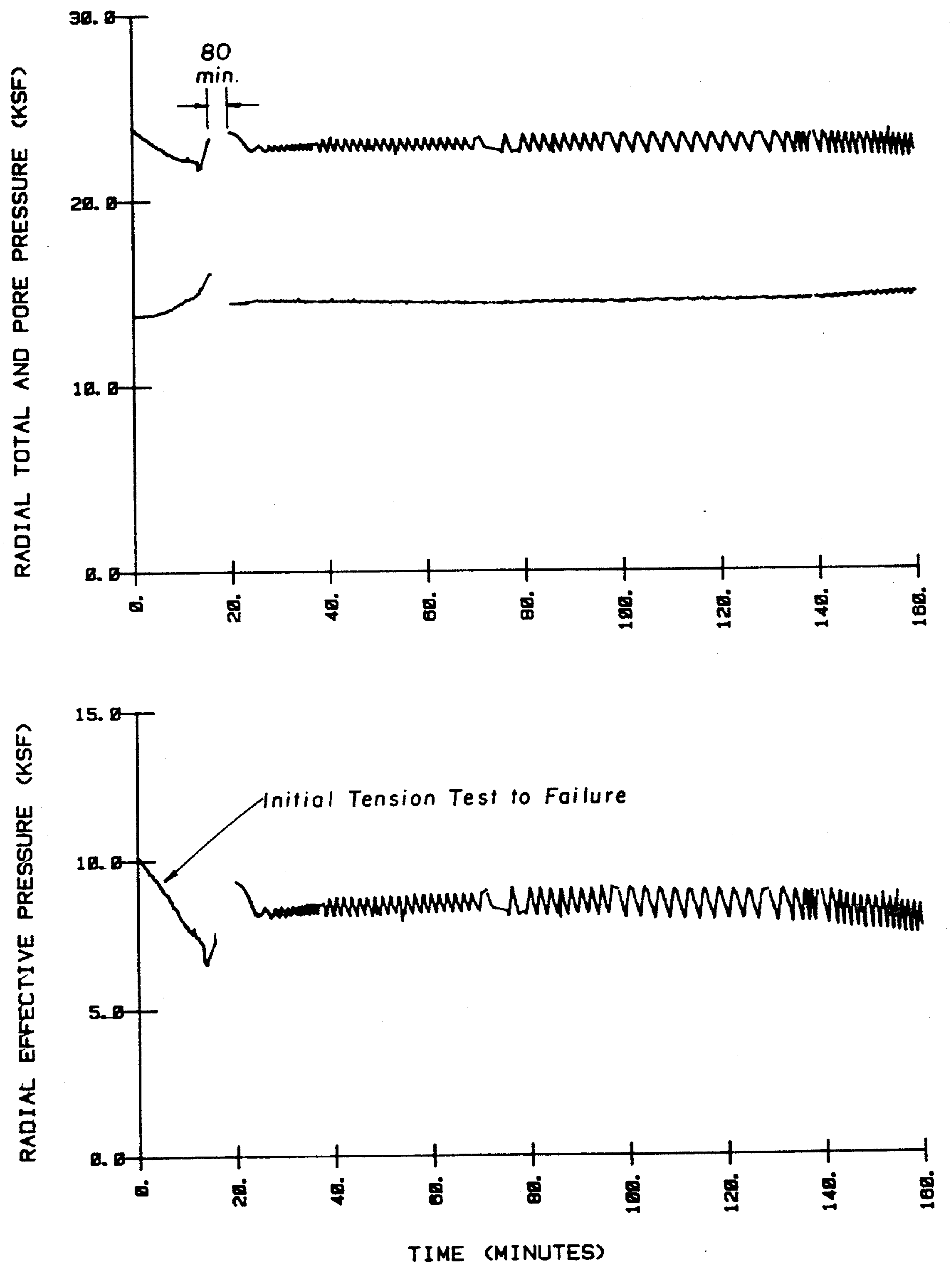
RESULTS OF THE LOAD TEST IMMEDIATELY AFTER DRIVING



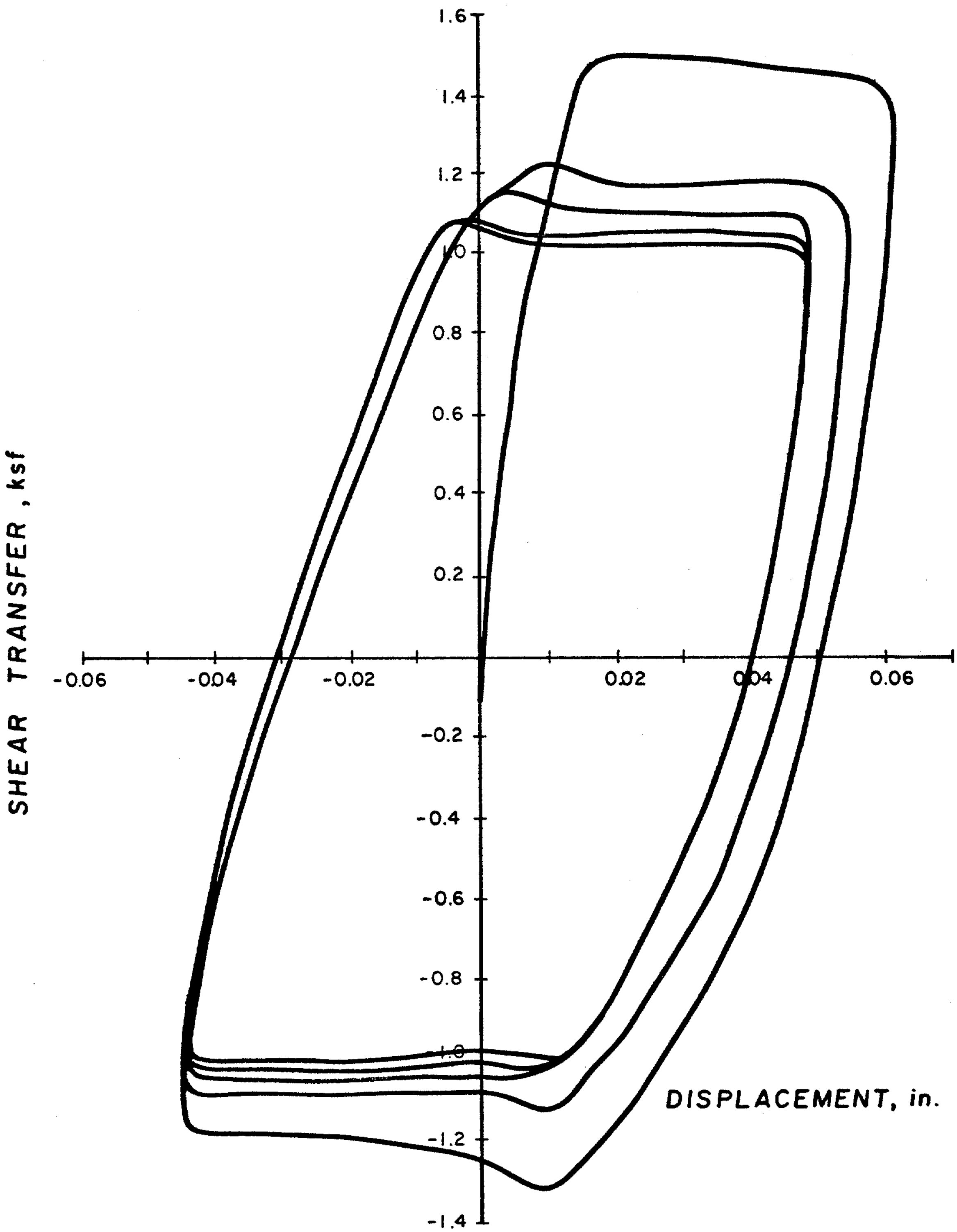
RESULTS OF THE TEST TO FAILURE AFTER CONSOLIDATION



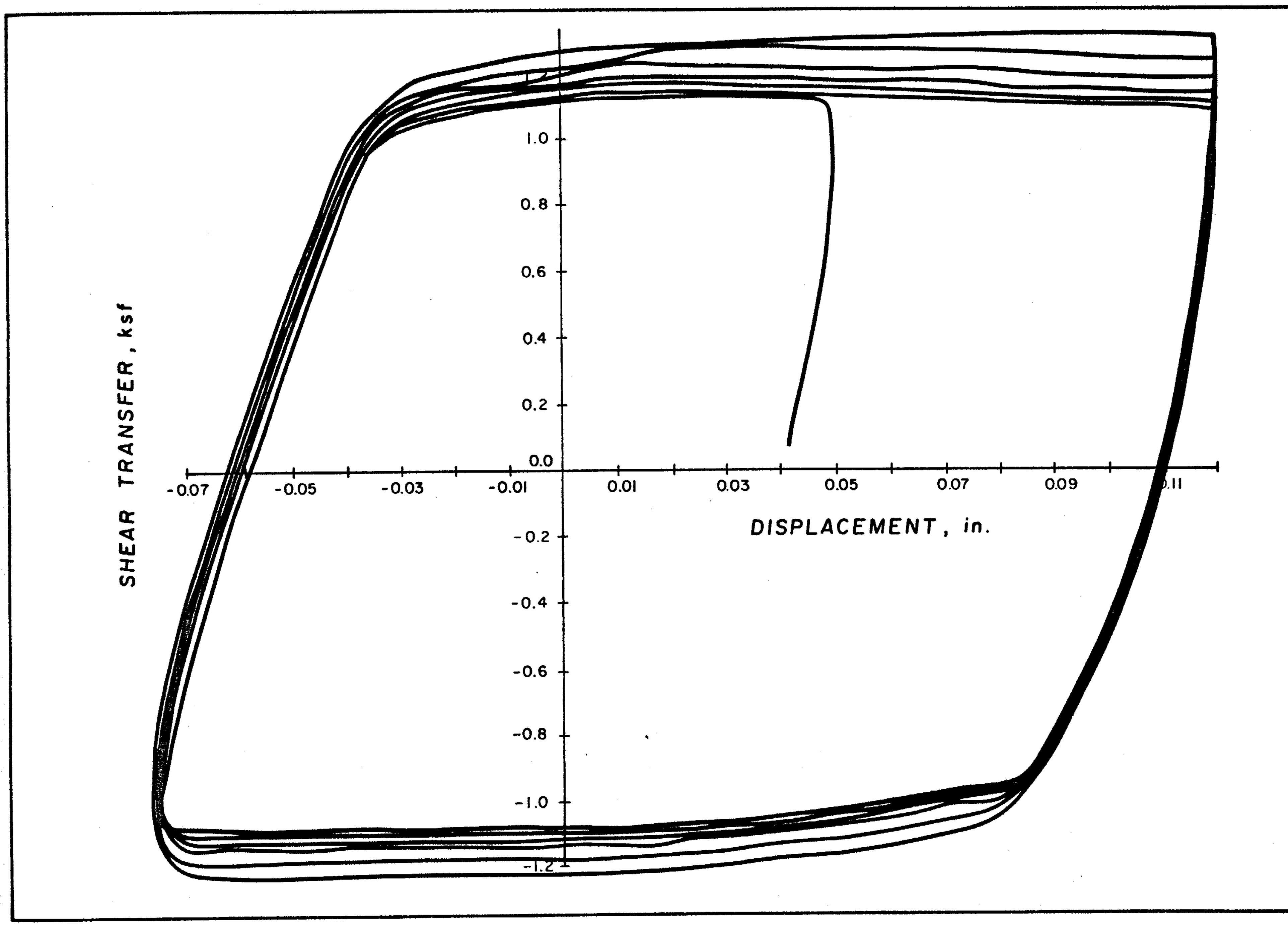
RESULTS OF THE ONE-WAY CYCLIC TENSION TESTS



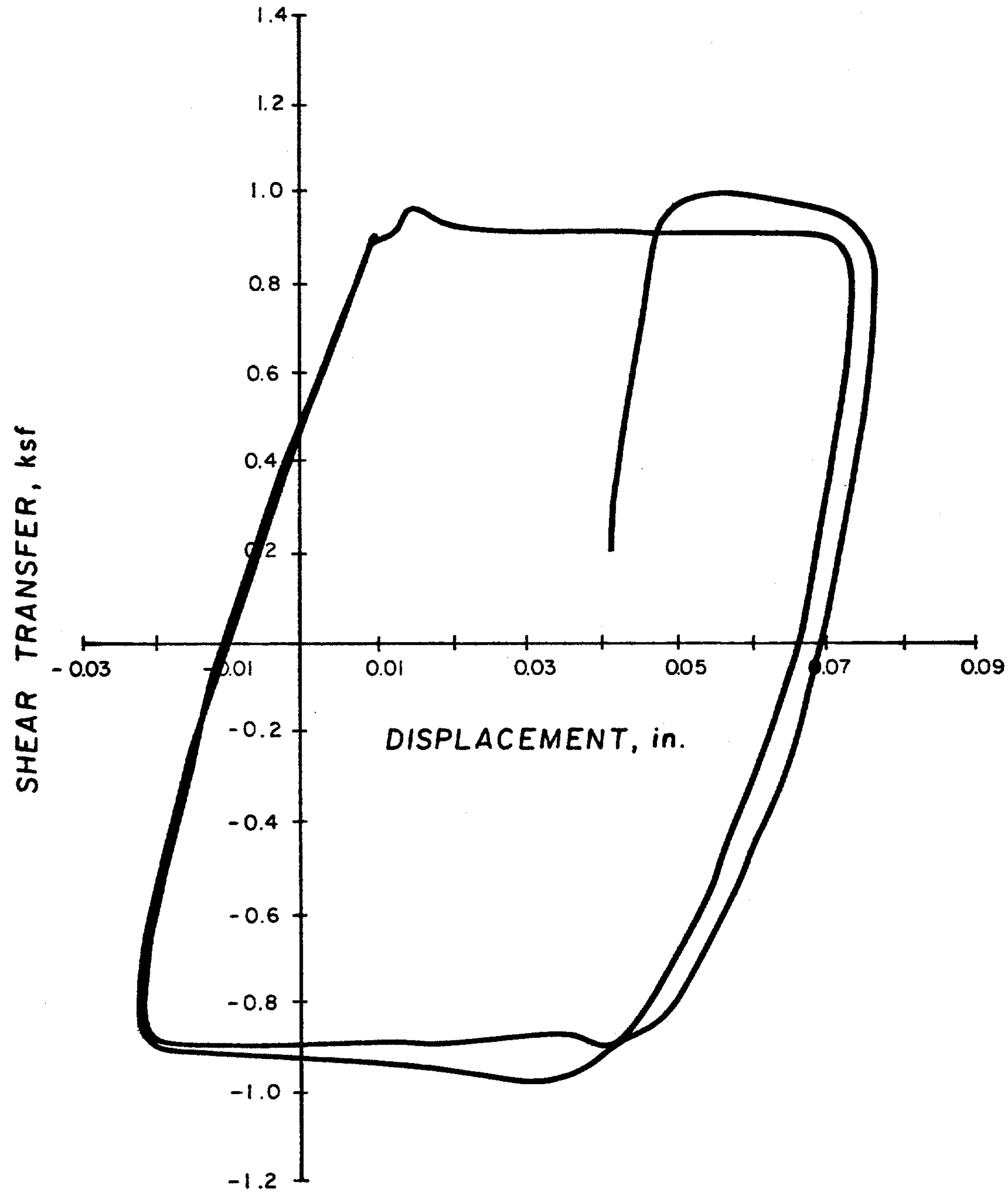
FLUCTUATIONS IN SOIL PRESSURE DURING THE ONE-WAY
CYCLIC TENSION TESTS



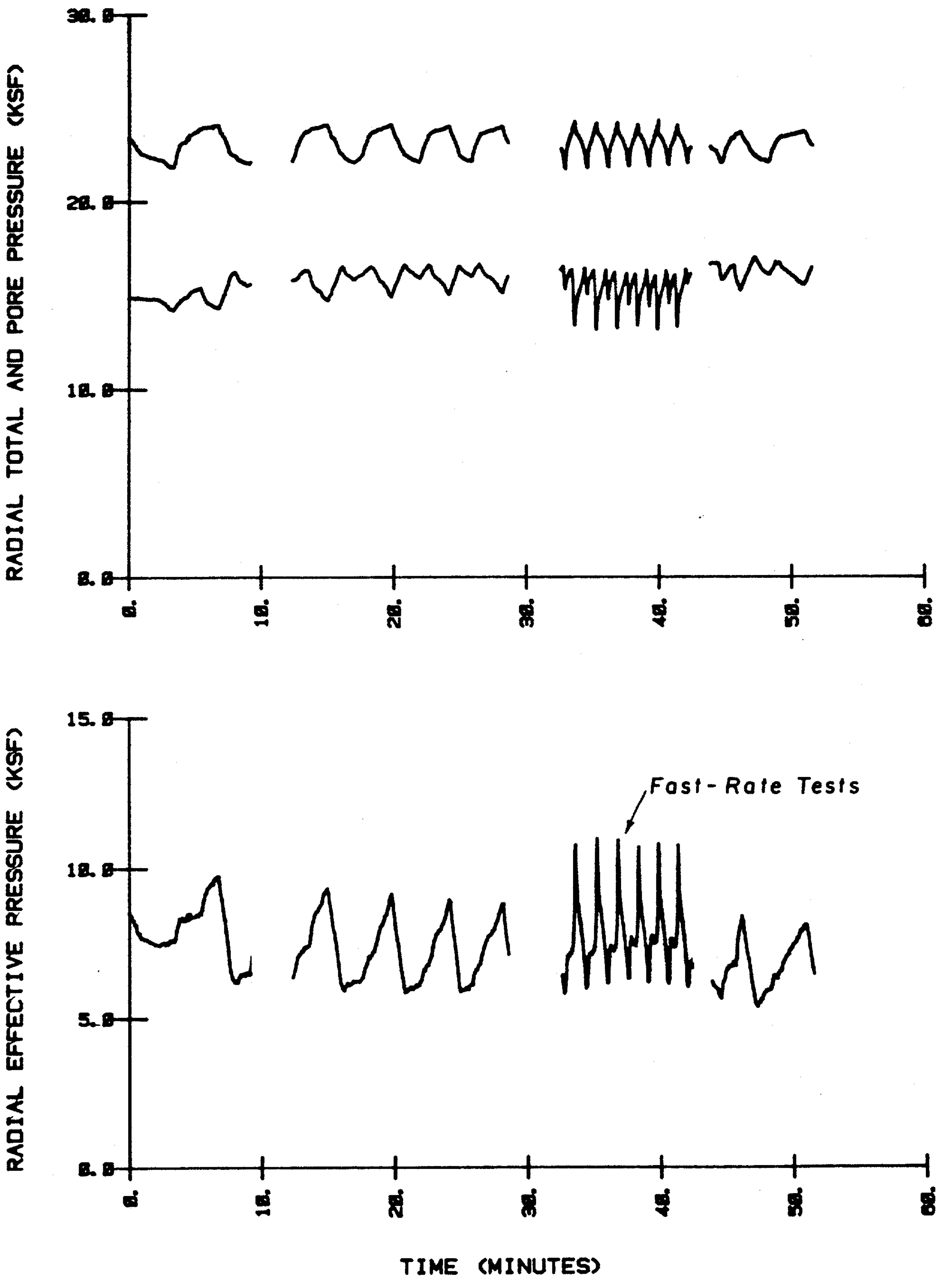
RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST



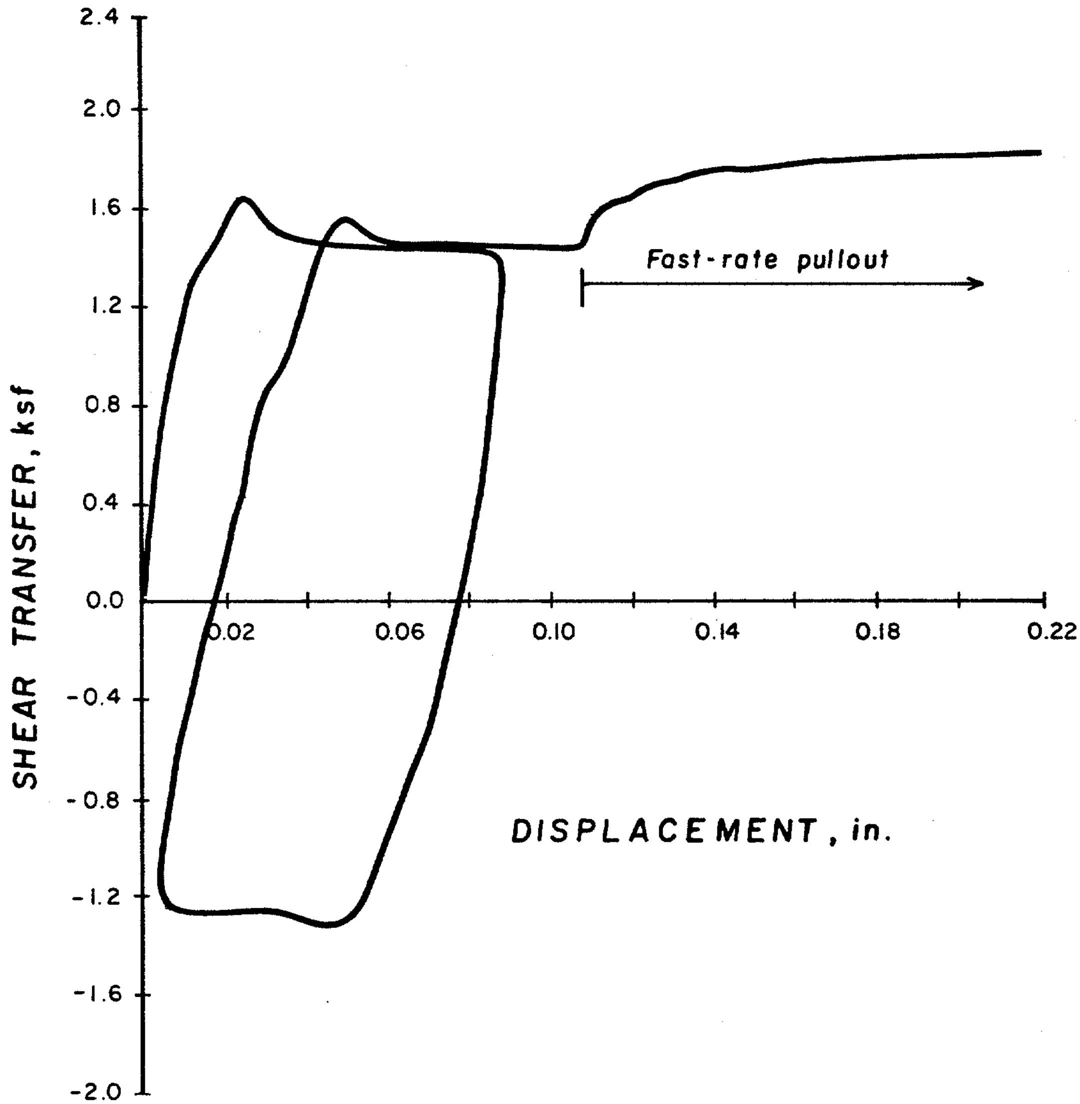
RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST



RESULTS OF THE REPEAT OF THE TWO-WAY CYCLIC TEST



FLUCTUATIONS IN SOIL PRESSURE DURING TWO-WAY CYCLIC LOADING



RESULTS OF THE LOAD TEST PERFORMED AFTER ADDITIONAL CONSOLIDATION

APPENDIX I: OPEN-END 3-IN. PROBE EXPERIMENT AT THE 160-FT DEPTH

RESULTS OF THE OPEN-END 3-IN. PROBE EXPERIMENT AT THE 160-FT DEPTH

The experiment performed with the open-end 3-in. probe was performed during the period from 1100 hours on 23 April until 1200 hours on 26 April.

The probe was installed by driving, with the impact force applied to the top of the N-rod string by a 300-lb casing hammer. The driving was begun at 11:19 and was completed at 11:31.

The subsequent variation in the soil pressures are shown in Plate I-1. The maximum total pressure after installation was 29.7 ksf; the maximum pore pressure was 22.3 ksf. It should be noted that the time scale in Plate I-1 begins one minute after driving; the values of maximum total and pore pressure are not shown in the plate because of the rapidity of the early decreases in both values of pressure.

Again, the open-end 3-in. probe did not indicate an initial value of radial effective pressure of the same magnitude as the full-displacement probes. Because of the sensitivity of the value of the initial pressure to slight variations in the degree of cavity expansion at the lower values, variations in the initial value should be expected.

As shown in the plate, a load test was performed 16 minutes after installation and then consolidation was allowed to proceed for 69 hours before performing the major series of load tests. Because of the schedule of experiments, the probe was removed upon completion of the major load test program.

The results of the load test performed 16 minutes after installation are shown in Plate I-2. The peak shear on the first failure was 0.48 ksf; the residual shear transfer after two reversals was 0.34 ksf.

As noted earlier, consolidation was then allowed to proceed for 69 hours. During this period, the radial total pressure decreased from 18.6 ksf to 16.8 ksf; the pore pressure decreased from 17.9 ksf to 12.0 ksf, indicating an increase in the radial effective pressure from 0.7 ksf to 4.8 ksf.

The rate of loading was returned to the slow rate, with the results shown in Plate I-8. The peak shear on the second failure in tension was 0.93 ksf; the residual shear transfer was 0.90 ksf. The rate of slip was 0.0018 in./sec.

The effects of rate on the limiting shear transfer during this experiment were thus approximately 26 percent per log cycle of increase in the rate of slip.

The fluctuations in the soil pressures during the two-way cyclic tests are shown in Plate I-9. The fluctuations are not as dramatic as those exhibited by the full-displacement probes, but follow the same general pattern, with the decreases inpore pressure, and the corresponding increases in the effective pressure, occurring during plastic slip at constant shear.

The probe was then removed from the boring, with one additional load test being performed. During the process of removing the probes, the hydraulic ram was used to free the cutting shoe, so that the drilling rig could then pull the string from the boring.

During the pullout of this probe, the rate of slip was varied, with the results shown in Plate I-10. The rate of slip was increased from 0.0022 in./sec to 0.027 in./sec, with the limiting shear transfer increasing from 0.94 ksf to 1.24 ksf, as shown in the plate. This again corresponds to a rate effect of approximately 29 percent per log cycle of increase in rate of slip.

The probe was then subjected to a slow monotonic loading to failure in tension with the results shown in Plate I-3. As shown in the plate, the peak shear transfer was 1.35 ksf, with a residual value of 1.16 ksf. Again, the value of 1.35 ksf is reasonably close to the value of the estimated undrained shear strength, 1.42 ksf.

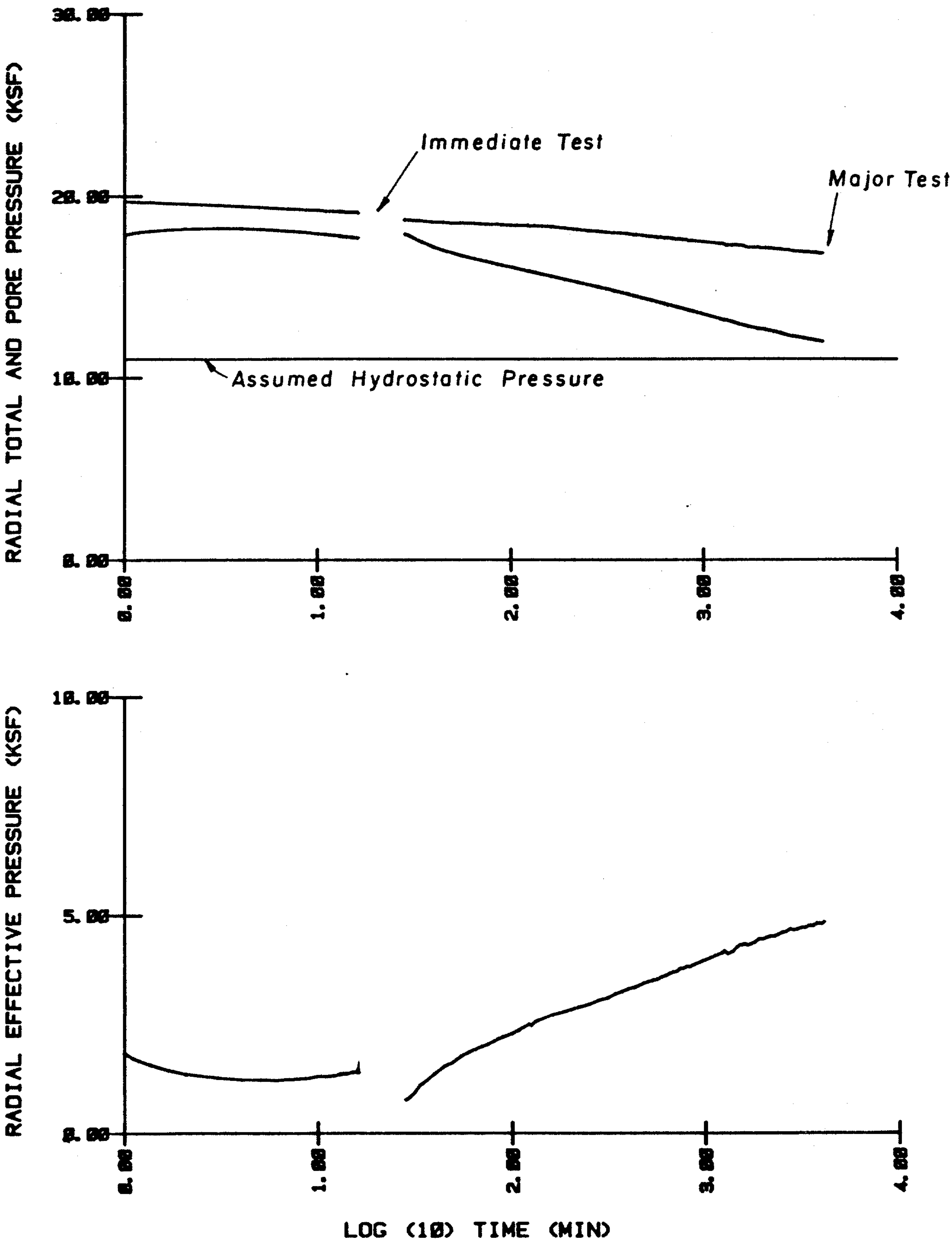
After completion of the initial load test, the pore pressures were allowed to equilibrate. After 27 minutes, the pore pressure returned to a value of 12.0 ksf from a value of 12.4 ksf at the end of the load test. The total pressure increased slightly, from 16.1 ksf to 16.3 ksf, indicating an increase in the radial effective pressure from 3.7 ksf to 4.3 ksf.

A series of one-way cyclic (repeated) tension loadings were then applied to the probe, with the results shown in Plate I-4. Again, a linear progression of accumulated permanent displacement occurs at values of shear transfer greater than 60 percent of the initial maximum shear; again, the rate of accumulation does not increase with the number of cycles until the value of the maximum load is equal to 100 percent of the previous maximum.

The fluctuations in the soil pressures during the one-way cyclic tension tests are given in Plate I-5, from which it can be seen that the one-way cyclic tension loading had little effect on the soil pressures.

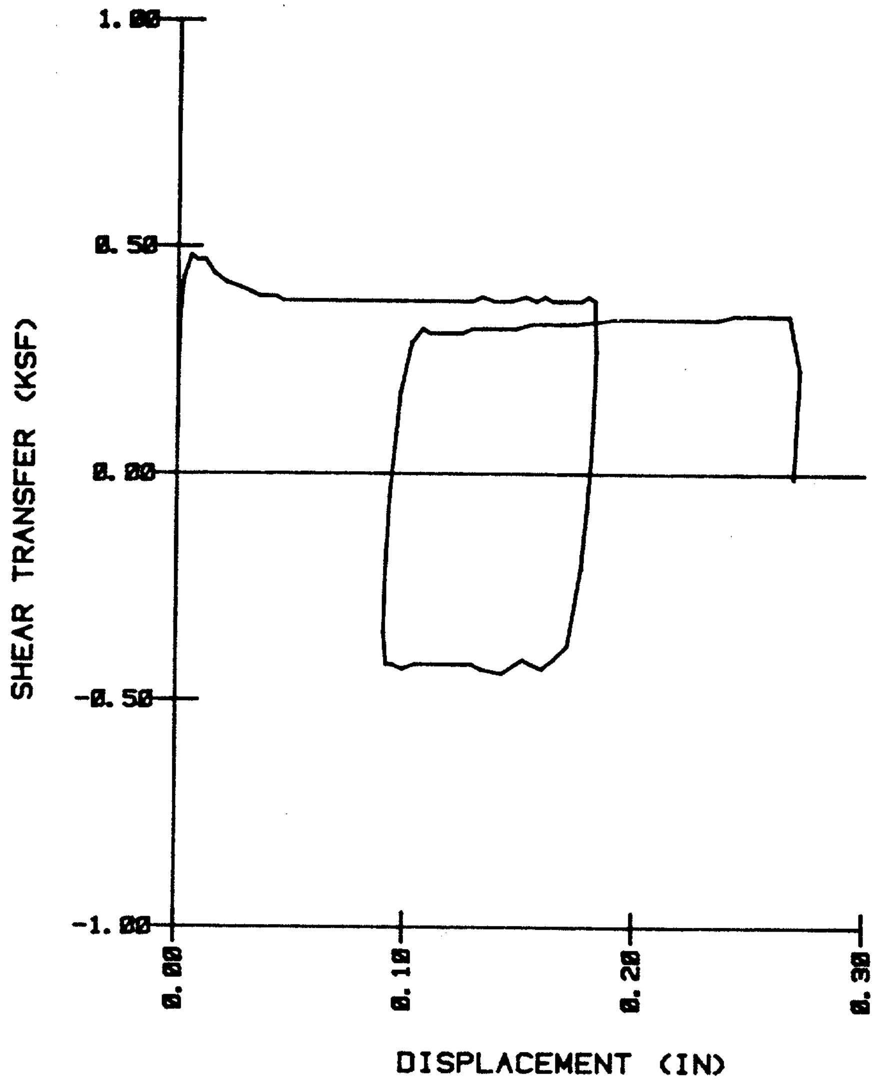
The probe was then subjected to a slow monotonic loading to failure in tension followed by four cycles of controlled-displacement two-way cycling. The results of these loadings are shown in Plate I-6. The maximum shear transfer on the first loading was 1.38 ksf. After five cycles of loading, the limiting shear transfer had decreased to 1.06 ksf. The irregular shapes of the curves is due to problems with a flow-control valve. The valve was later replaced.

The rate of loading was then increased, with the results shown in Plate I-7. The limiting shear transfer on the fifth cycle was 1.17 ksf, with a rate of plastic slip of 0.0267 in./sec.

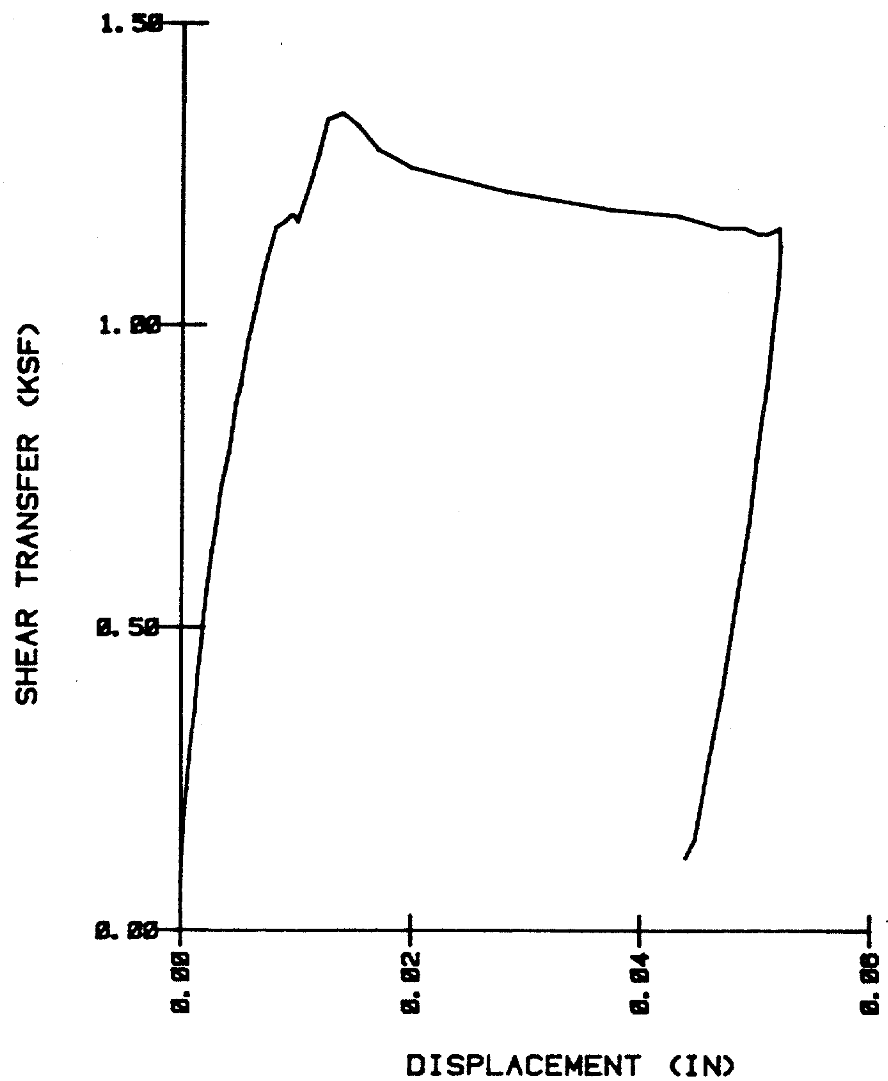


VARIATIONS IN SOIL PRESSURE DURING CONSOLIDATION

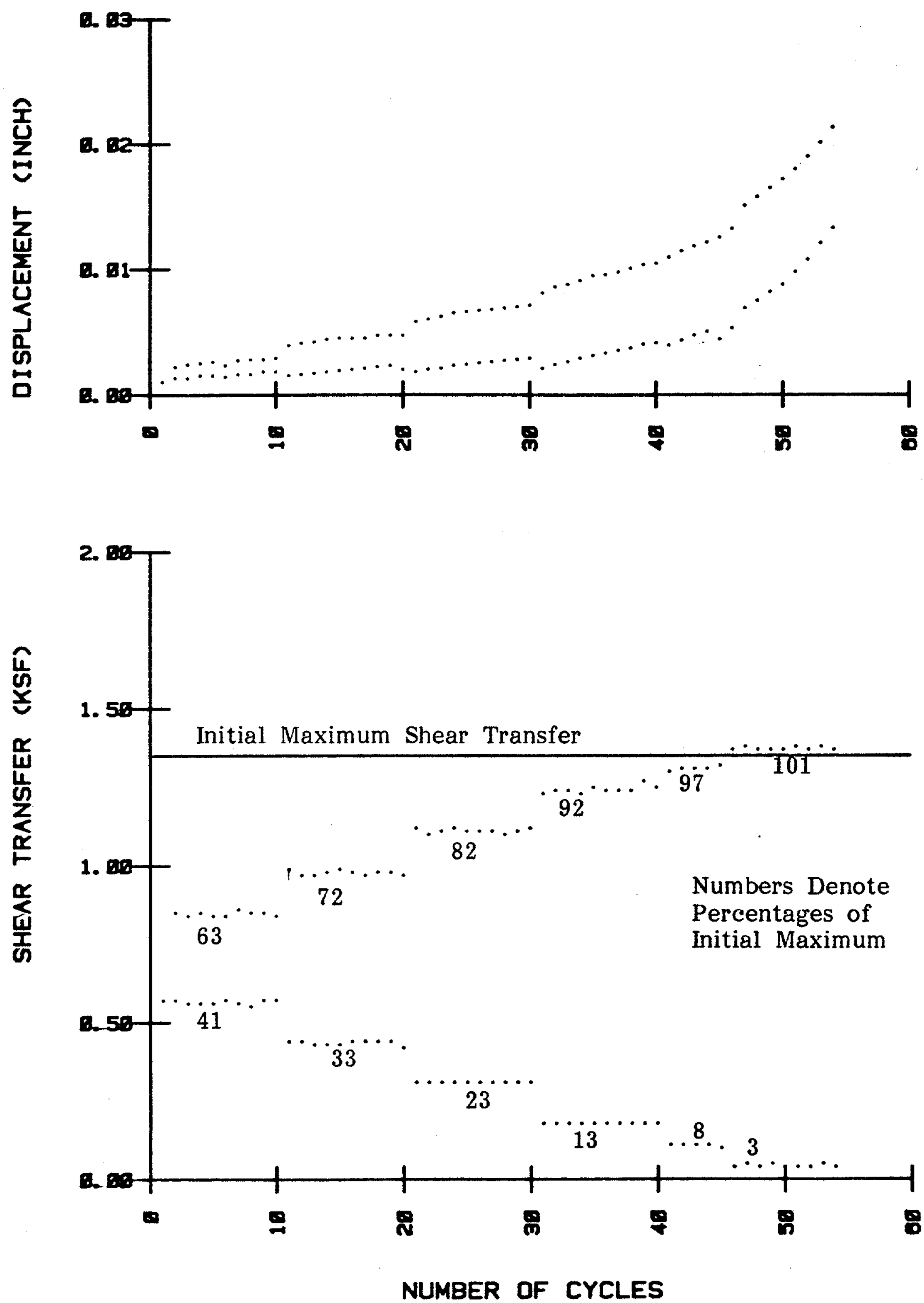
The Earth Technology Corporation



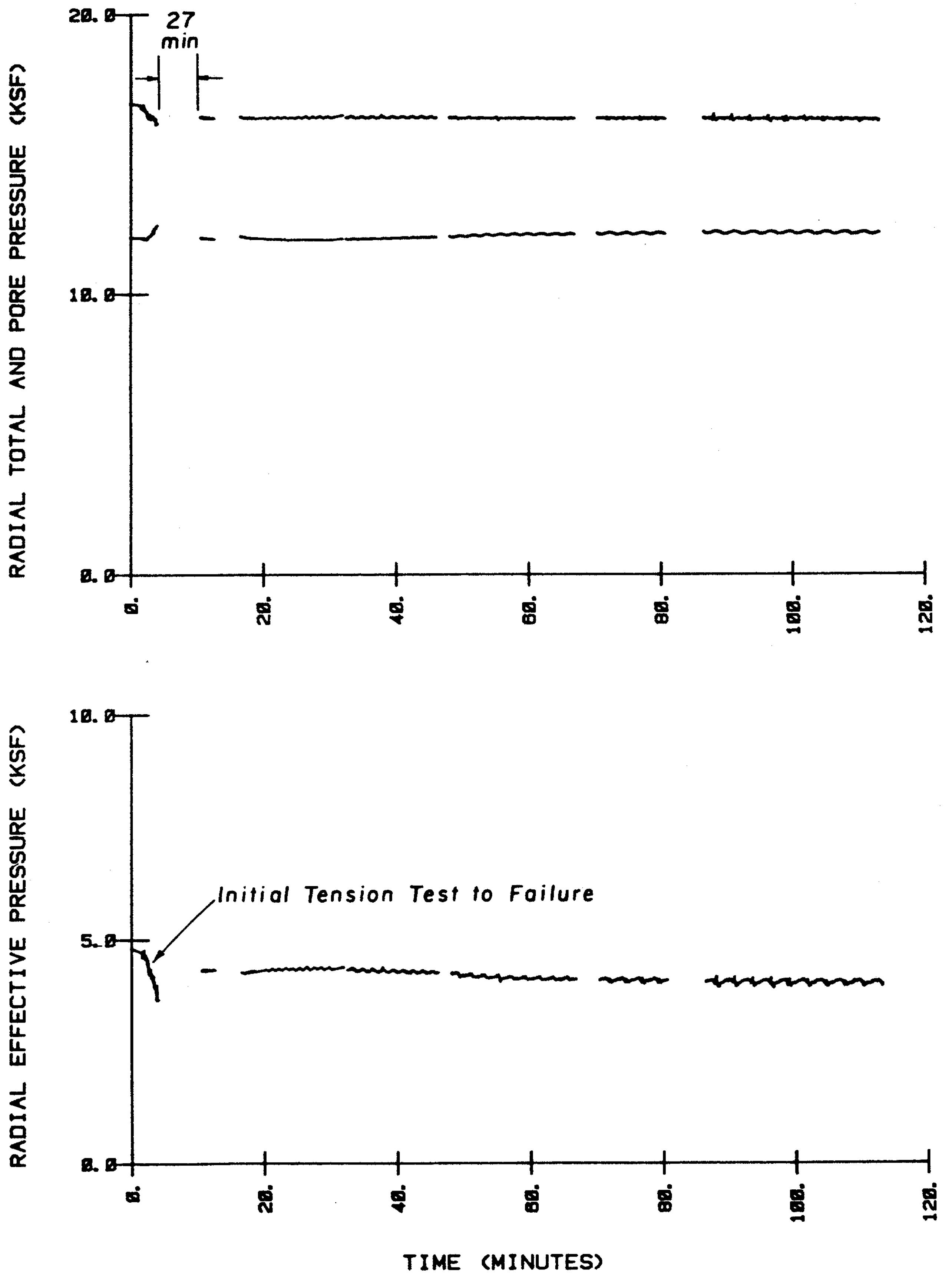
RESULTS OF THE LOAD TESTS IMMEDIATELY AFTER DRIVING



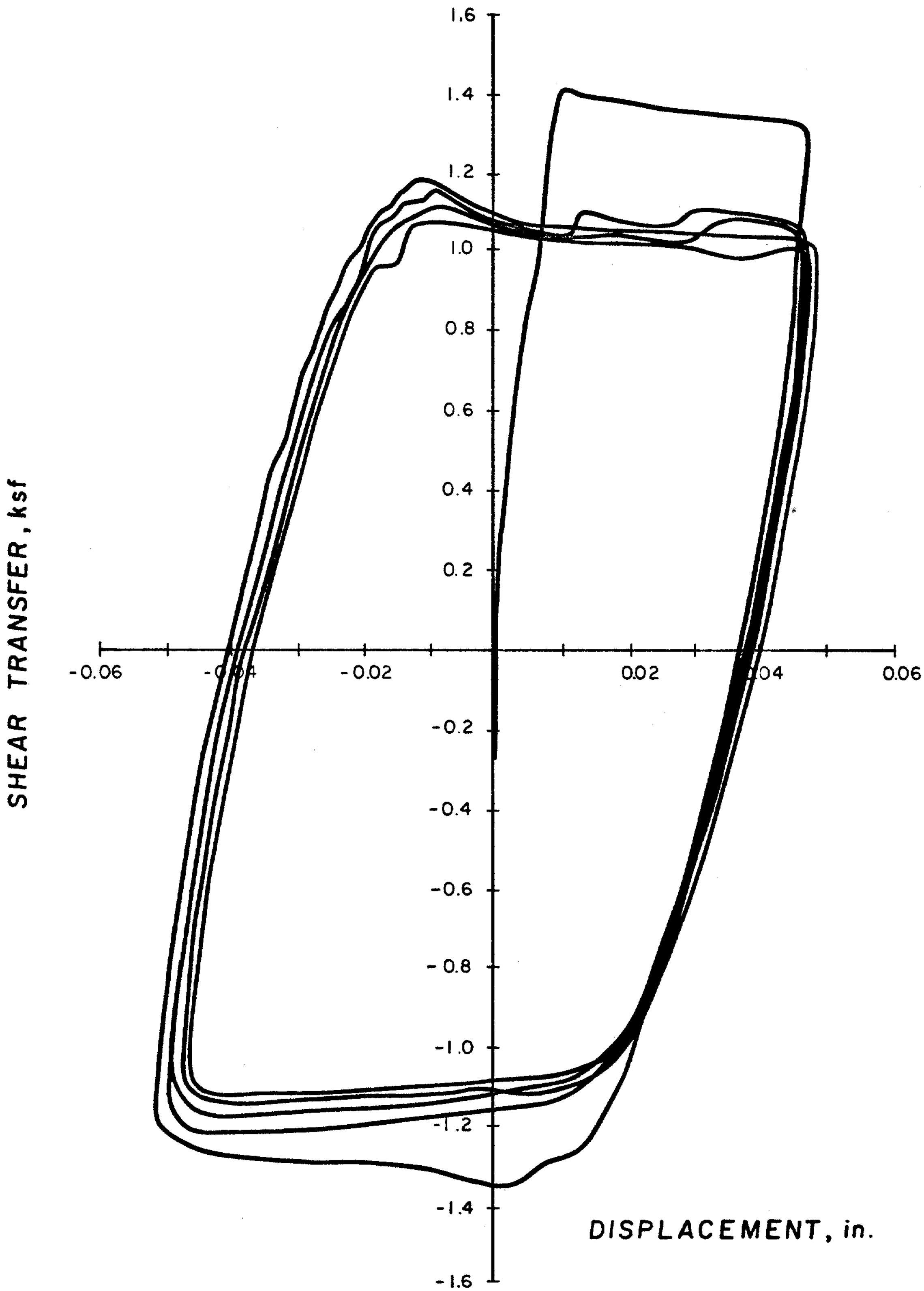
RESULTS OF THE INITIAL LOAD TEST AFTER CONSOLIDATION



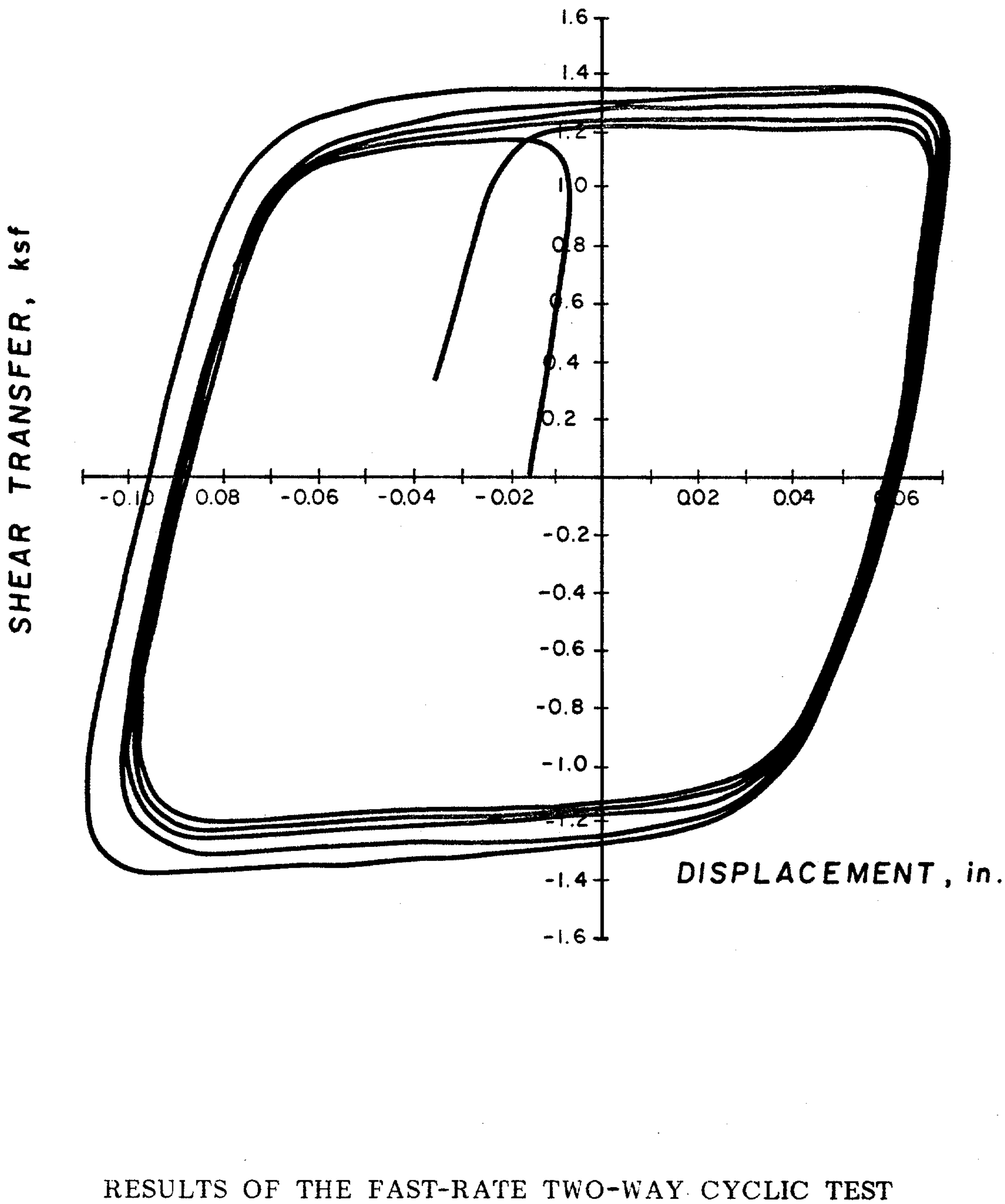
RESULTS OF THE ONE-WAY CYCLIC TENSION TESTS



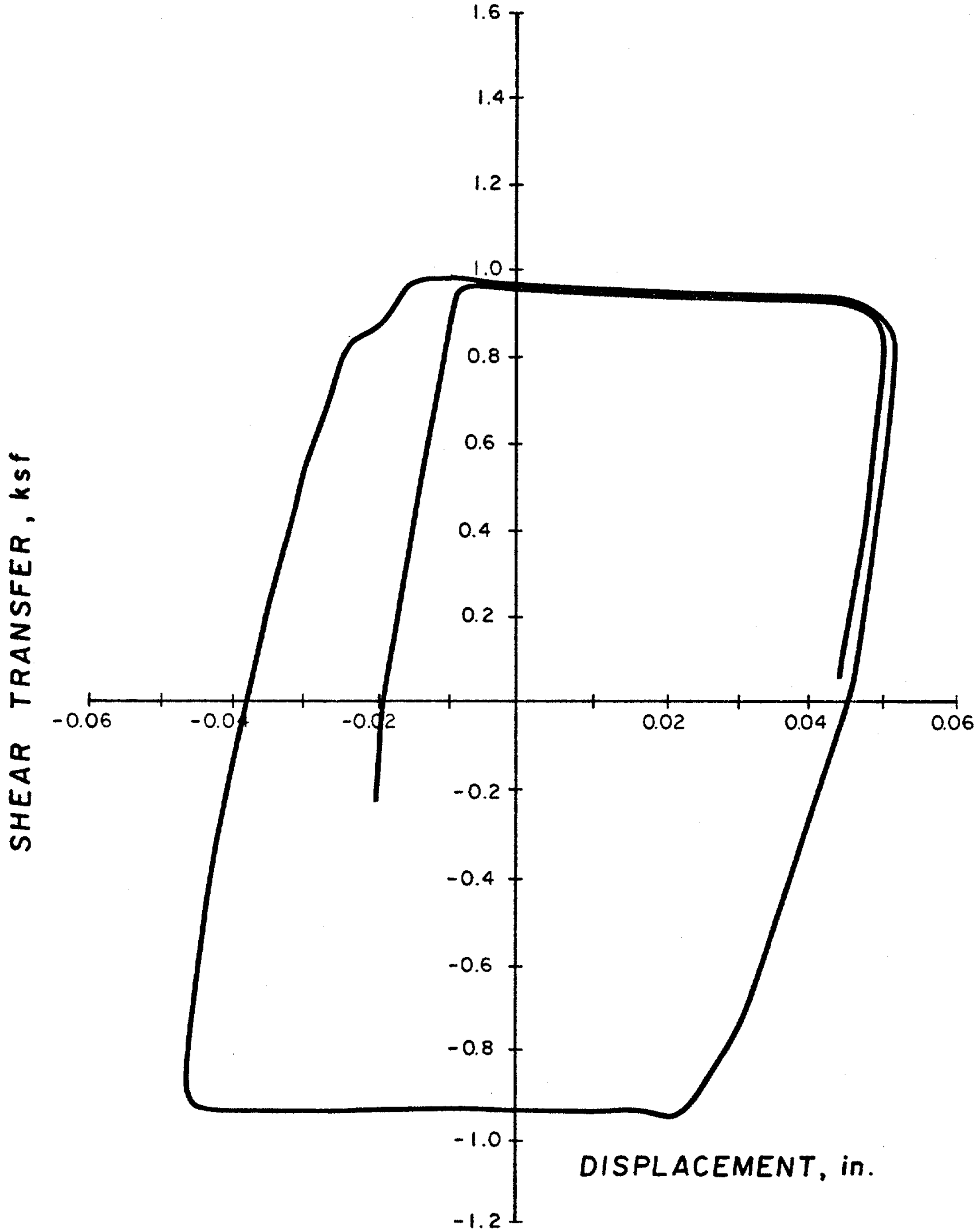
FLUCTUATIONS IN SOIL PRESSURE DURING ONE-WAY CYCLIC TENSION TESTS



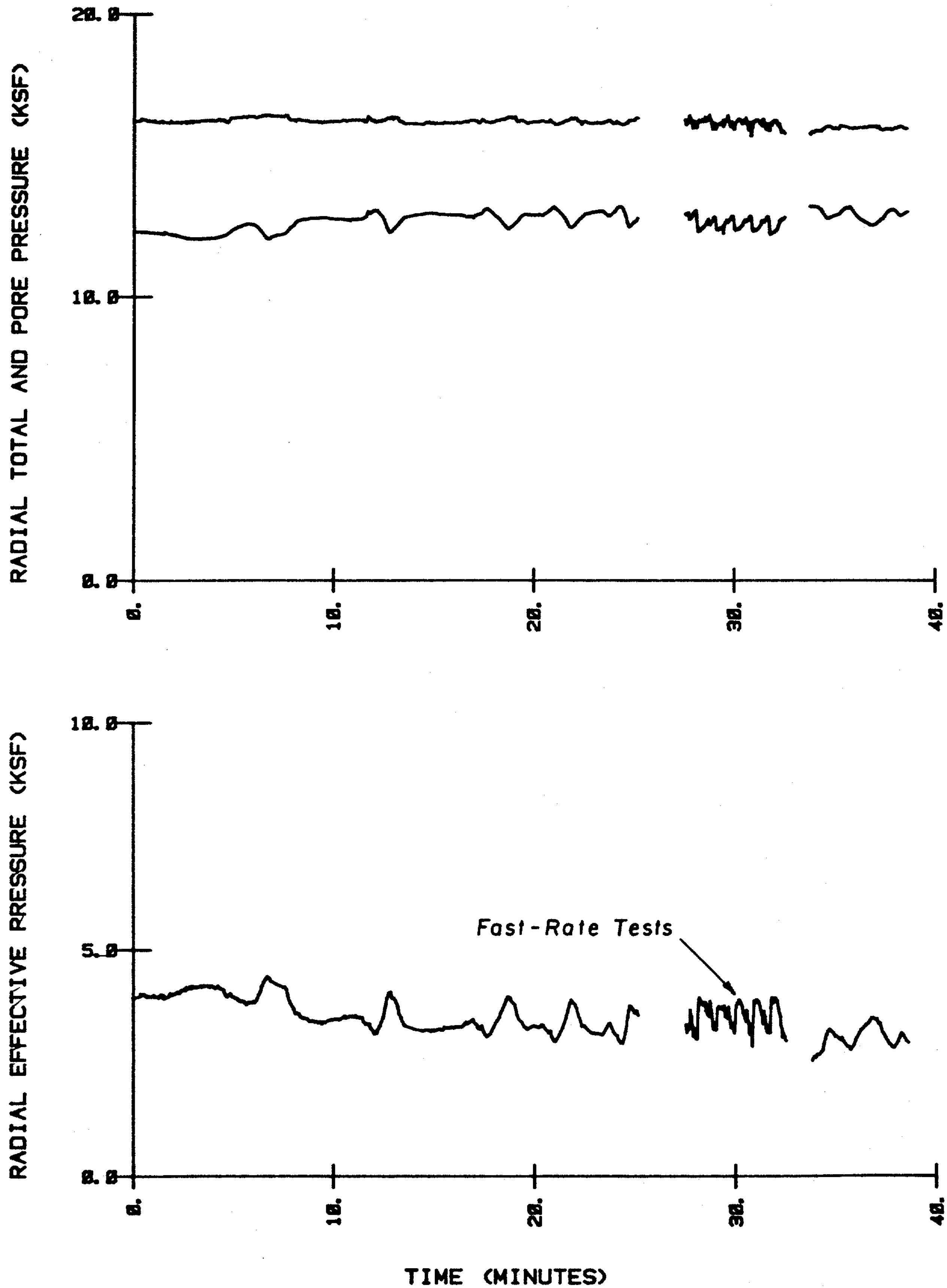
RESULTS OF THE INITIAL TWO-WAY CYCLIC TESTS



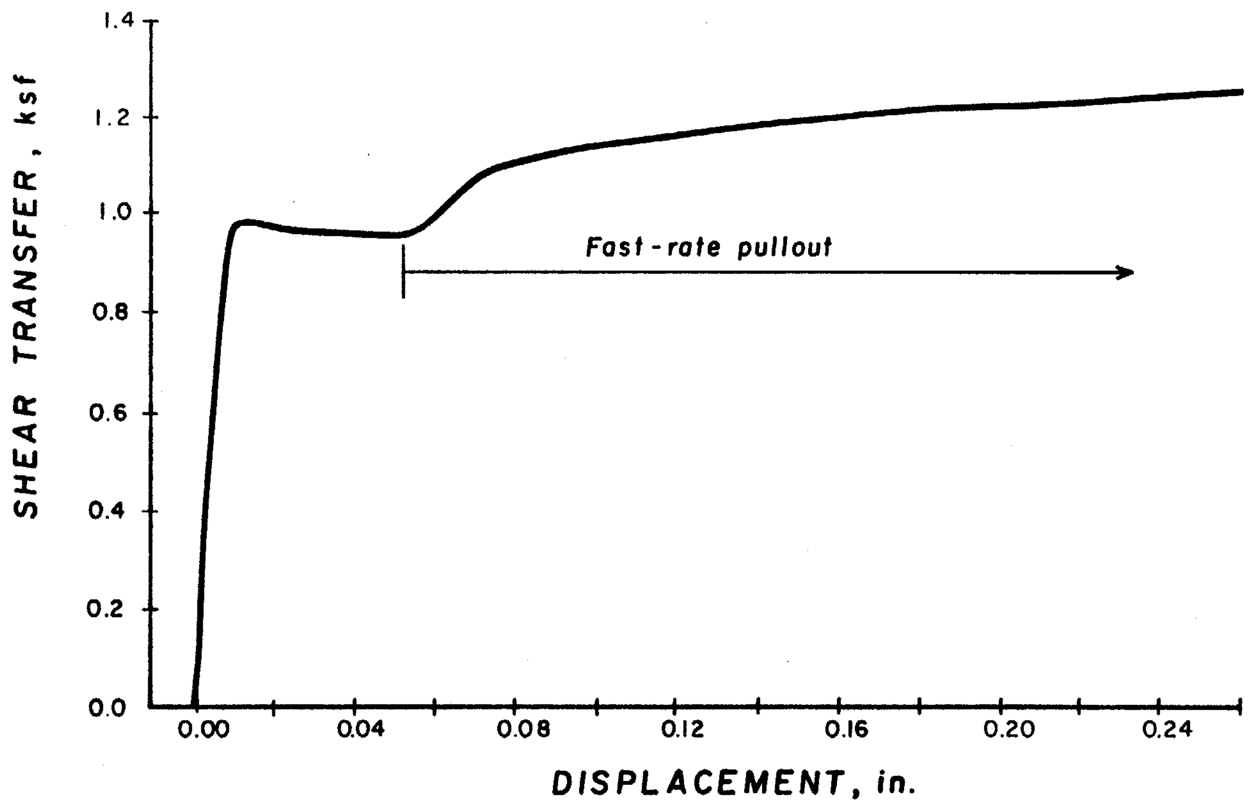
RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST



RESULTS OF THE REPEAT OF THE TWO-WAY CYCLIC TEST



FLUCTUATIONS IN SOIL PRESSURE DURING TWO-WAY CYCLIC LOADING



RESULTS OF THE VARIABLE-RATE TEST DURING REMOVAL OF THE PROBE

APPENDIX J: X-PROBE EXPERIMENT AT THE 210-FT DEPTH

RESULTS OF THE X-PROBE EXPERIMENT AT THE 210-FT DEPTH

The experiment with the X-probe at the 210-ft depth was performed during the period from 1630 hours on 19 April and 160 hours on 20 April.

The probe was installed by pushing it into place using the drawdown on the drilling rig. The final push was made at 16:58.

The variation in soil pressures during consolidation are shown in Plate J-1. The maximum total pressure after insertion was 29.3 ksf, with a maximum pore pressure of 28.4 ksf. Both the total and the pore pressure decreased rapidly, being 26.0 ksf and 21.8 ksf, respectively, at the time of the first load test, 11 minutes after insertion.

The results of the first load test are given in Plate J-2. The maximum shear during this loading was 2.03 ksf.

Consolidation was allowed to proceed for 16 hours, during which time the total pressure decreased only slightly, from 24.6 ksf to 24.2 ksf, and the pore pressure decreased from 21.1 ksf to 14.2 ksf. The corresponding increase in the radial effective pressure was from 3.5 ksf to 10.0 ksf. The behavior shown in Plate J-1 indicates that the soil at this depth was not a plastic clay, since the consolidation was very rapid, and does not fit the patterns established in the clay at the shallower dpeths.

The results of the initial loading to failure after consolidation are shown in Plate J-3. Although the radial effective pressure increased from 3.5 to 10.0 ksf during consolidation, the maximum shear transfer recorded during this test was only 1.94 ksf, some 0.1 ksf (5 percent) less than that during the first test. Although the soil type is not definitely established, the rate of consolidation suggests that the material was a silt. It is interesting to note that the measured shear transfer is akin to the often-quoted limit value of 2 ksf for the shear transfer in silts.

The total radial pressure at the end of the test was 23.6 ksf. After a period of 20 minutes, the total pressure was 23.5 ksf. During the same period, the pore pressure decreased from 15.6 ksf to 14.2 ksf, indicating an increase in the radial effective pressure from 8.0 ksf to 9.3 ksf.

The probe was then subjected to a series of one-way cyclic tension tests, with the results summarized in Plate J-4. As shown in the plate, the behavior observed during this series of tests also differed from that observed earlier no progressive upward displacements were observed until the load level had reached a vluae of 90 percent of the initial maximum shear transfer. At this load level, the rate of accumulation of permanent displacement rapidly accelerated, with a tensile failure occurring on the tenth cycle.

The fluctuations in soil pressure during the cyclic tension tests are shown in Plate J.-5.

The probe was then subjected to a slow monotonic loading to failure in tension with the results shown in Plate J-6. The maximum shear transfer during this loading was 1.80 ksf; only slightly greater than the value of 1.75 ksf which had resulted in the progressive pullout, and less than the initial maximum value of 1.94 ksf.

The probe was then subjected to five cycles of controled-displacement two-way cyclic loading, with the results shown in Plate J-7. The initial loading is also shown in Plate J-6. After five cycles, the shear transfer was reduced only slightly, with a peak shear transfer of 1.8 ksf, and a residual shear transfer of 1.47 ksf.

The rate of loading was then increased, with the results shown in Plate J-8. After four cycles, the displacement limits were increased, so that the digital system could obtain more samples. The results are shown in Plate J-9. On the third cycle, the maximum shear transfer was 1.62 ksf, with a slip rate of 0.0381 in./sec.

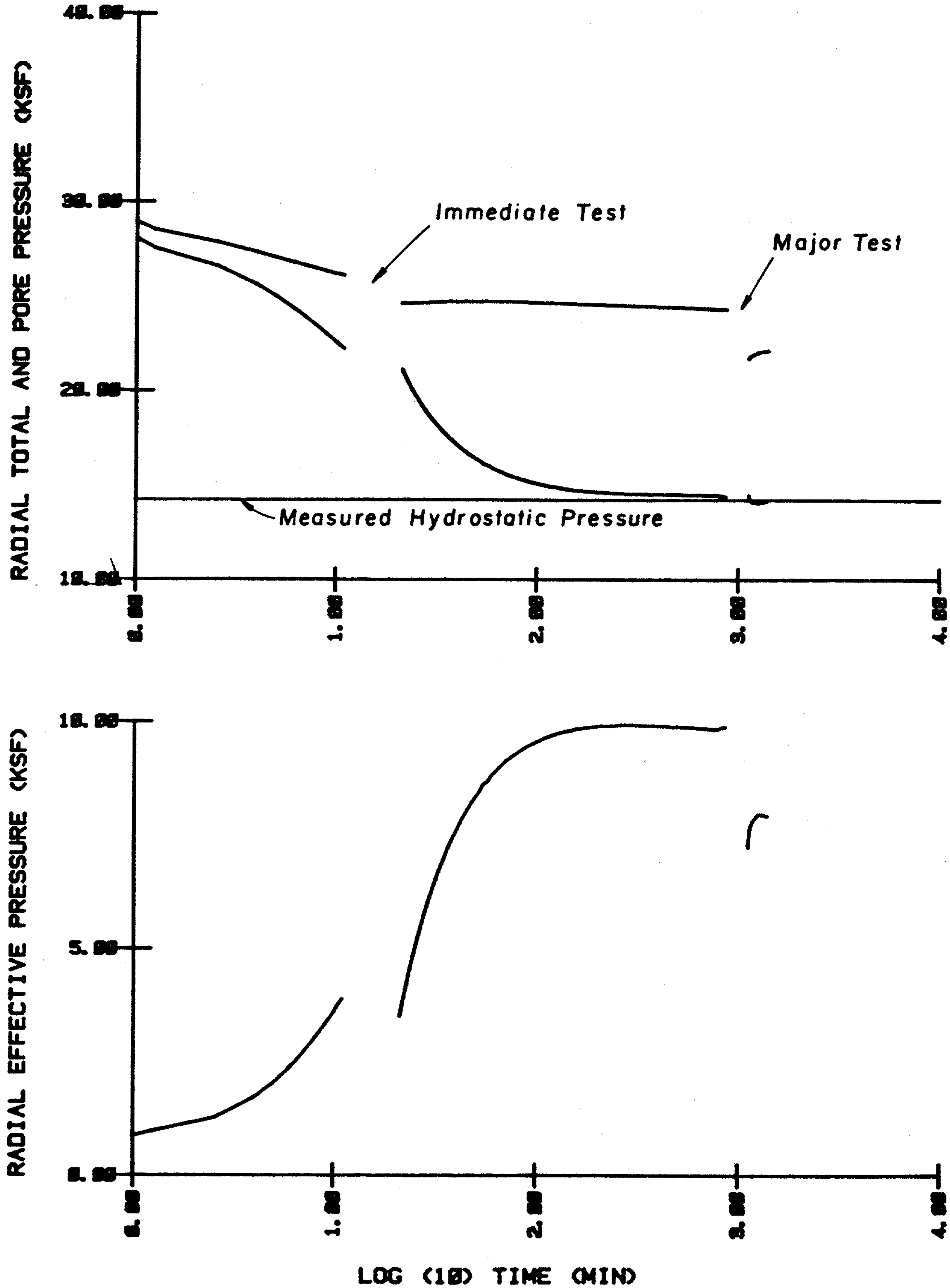
The load rate was then reduced, with the results shown in Plate J-10. The peak shear transfer during the tension loading was 1.86 ksf; the residual shear was 1.49 ksf.

The fluctuations in the soil pressures during the two-way cyclic tests are shown in Plate J-11. A comparison of this plate with those presented earlier shows that the pressure fluctuations were much larger in magnitude than those observed in the clay soil. The maximum recorded effective stress value was almost 18 ksf, an increase of almost 9 ksf above the value of 9.1 ksf prior to beginning the two-way cyclic tests, and more than 10 ksf greater than the value of 7.2 ksf which was recorded under quiescent conditions following the cyclic tests.

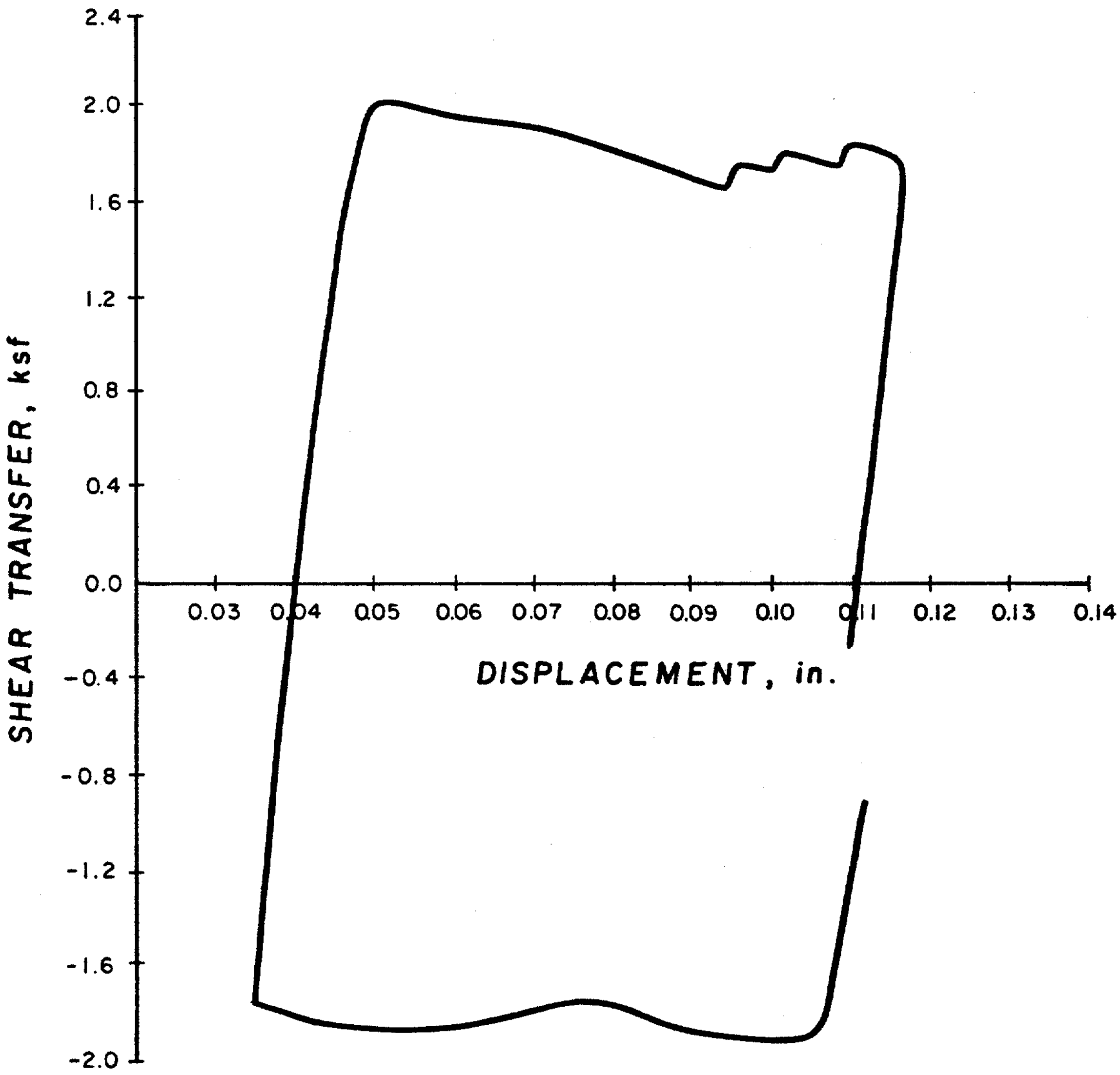
The probe was then left undisturbed for a period of about 5 hours, during which time the total radial pressure increased slightly, from 21.7 ksf to 22.1 ksf. The pore pressure decreased from 14.5 ksf to 14.2 ksf, indicating an increase in the radial effective pressure from 7.2 ksf to 7.9 ksf.

The probe was then loaded to failure in tension, followed by one reversal, with the results shown in Plate J-12. After the second reversal, the rate of loading was increased, with the resulting increase in shear transfer shown in the figure. The quasi-static behavior had not changed appreciably, with a peak shear on the first loading of 1.92 ksf, with a residual shear transfer of 1.50 ksf.

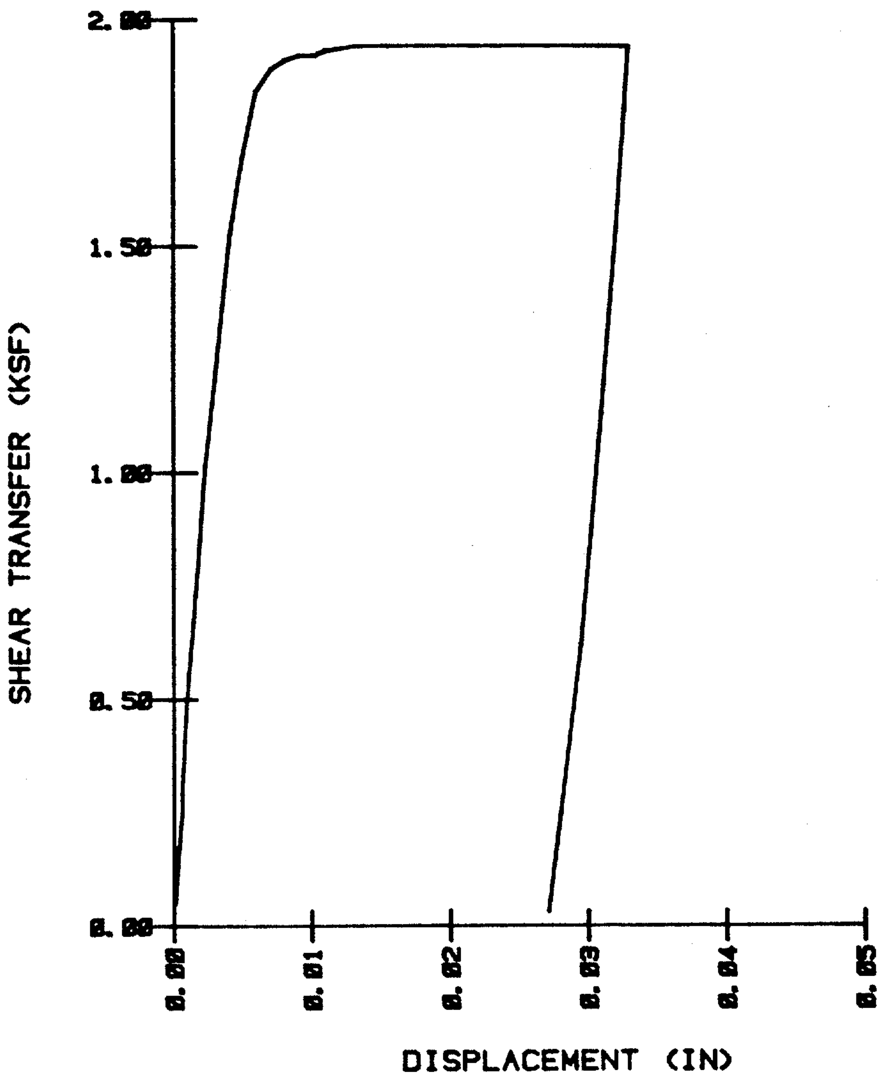
During final pullout of the probe, the fast rate of displacement resulted in a substantial increase in the residual value of the load transfer. However, since the rate of pullout did not stabilize before the maximum travel of the LVDT was reached, it is not possible to compute the rate of loading factor.



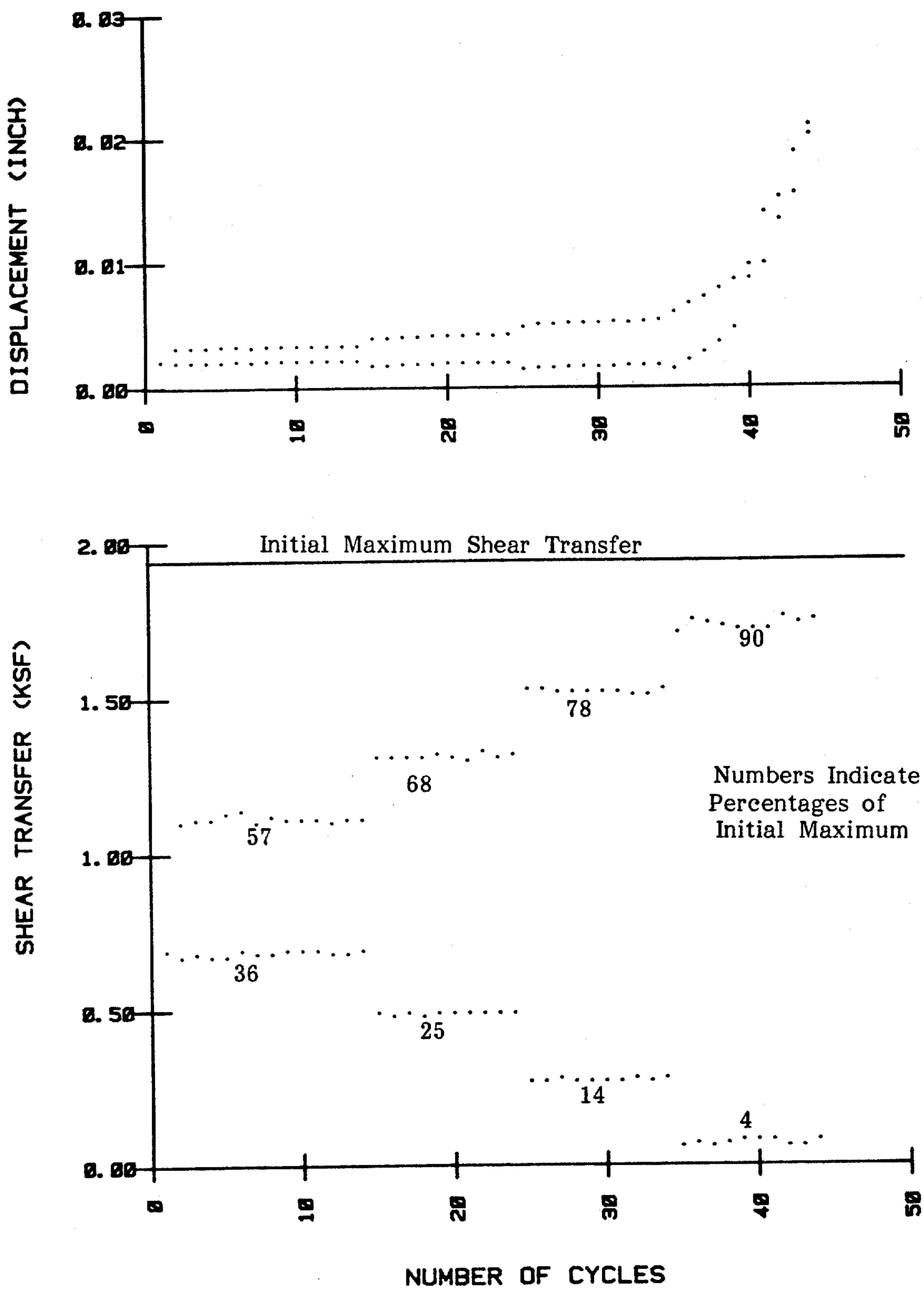
VARIATION IN SOIL PRESSURES DURING CONSOLIDATION



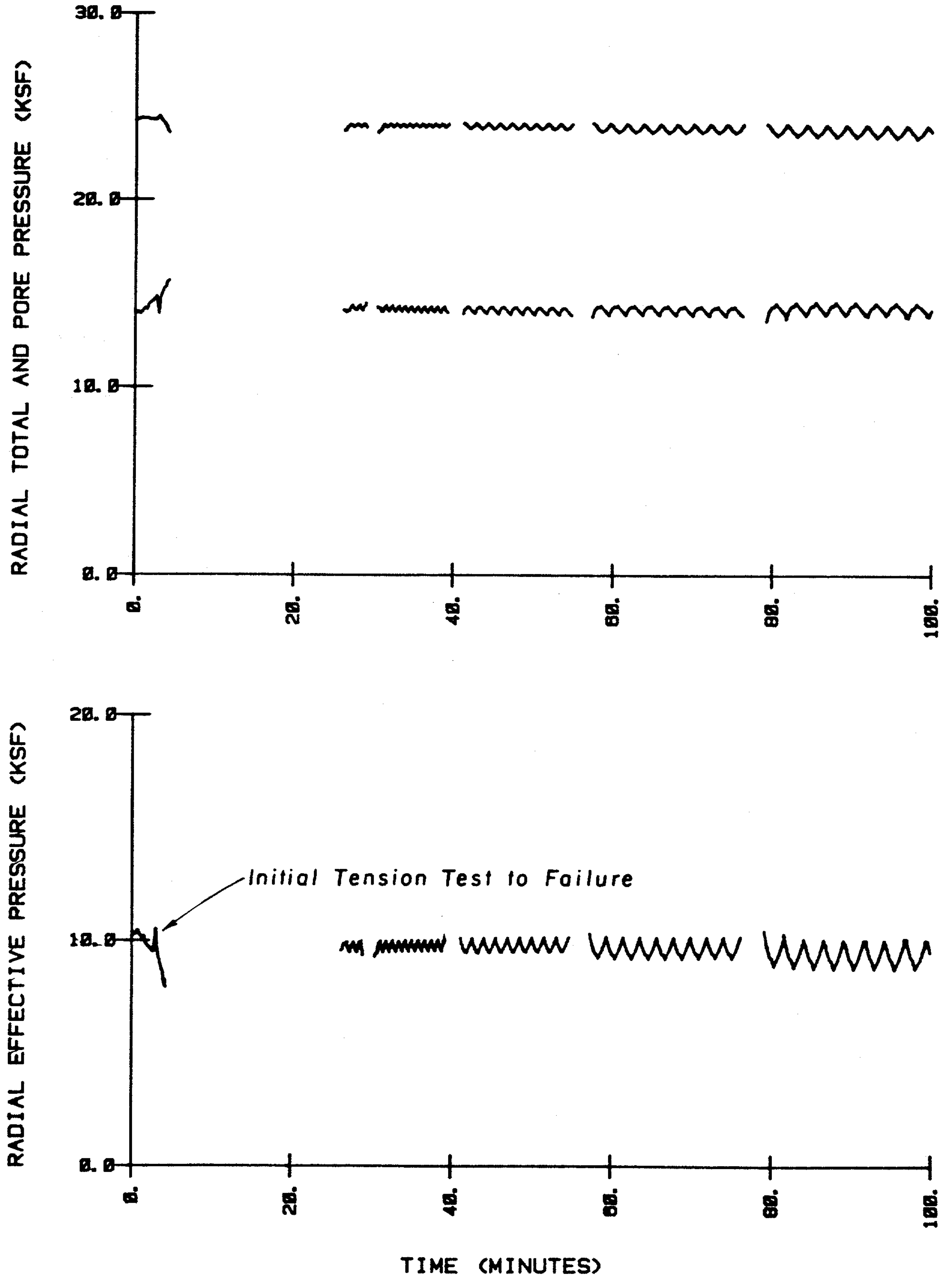
RESULTS OF THE IMMEDIATE LOAD TESTS



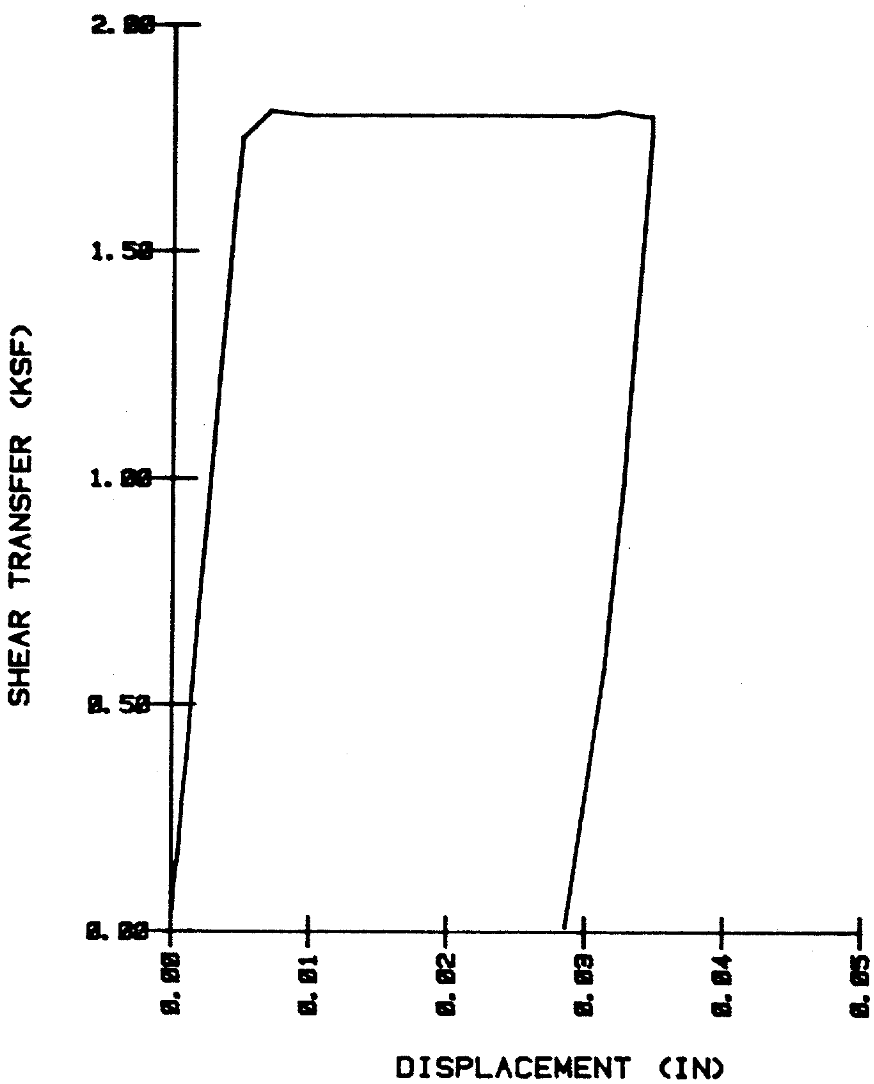
RESULTS OF THE TENSION TEST AFTER CONSOLIDATION



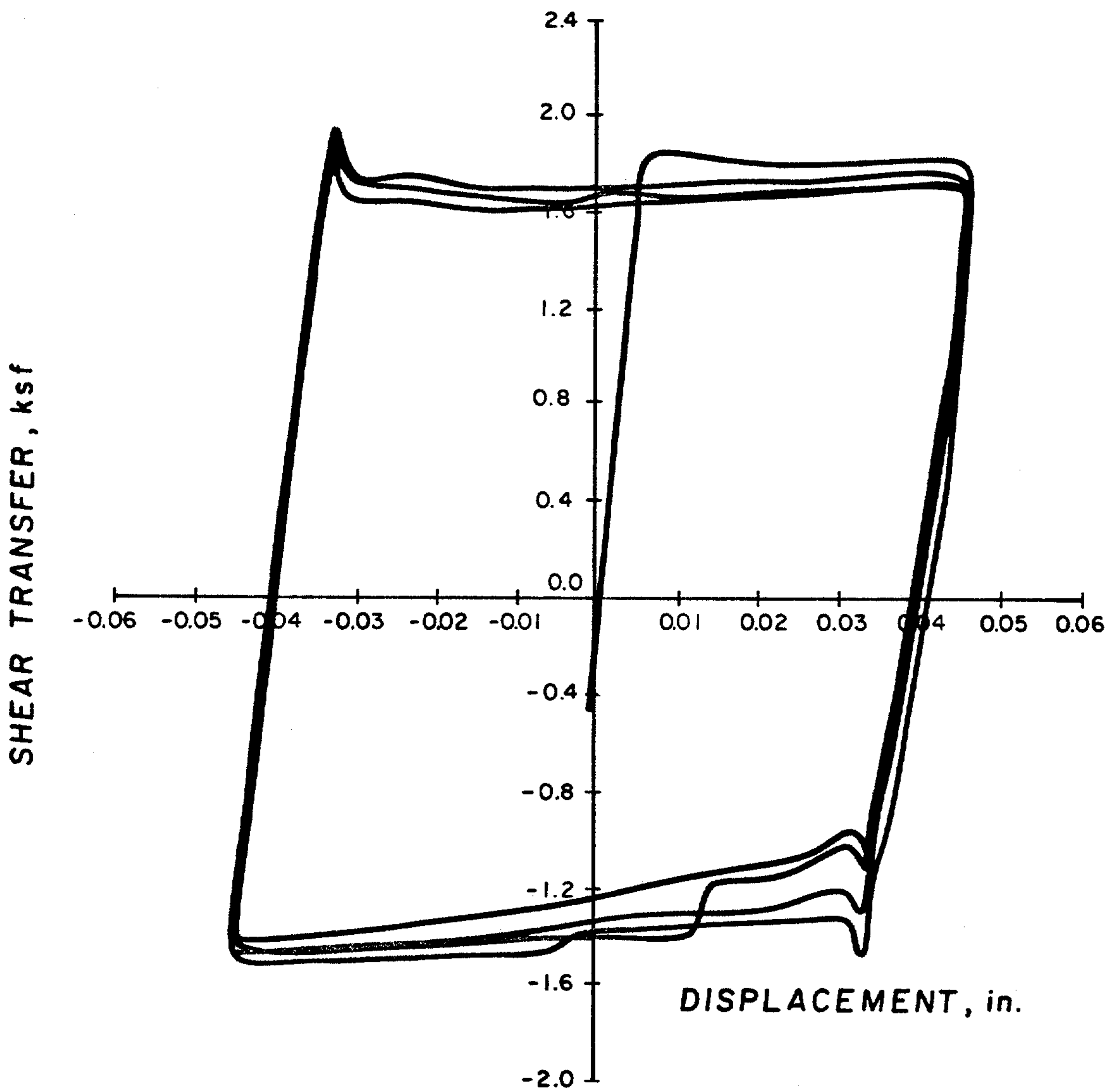
RESULTS OF THE ONE-WAY CYCLIC TENSION TESTS



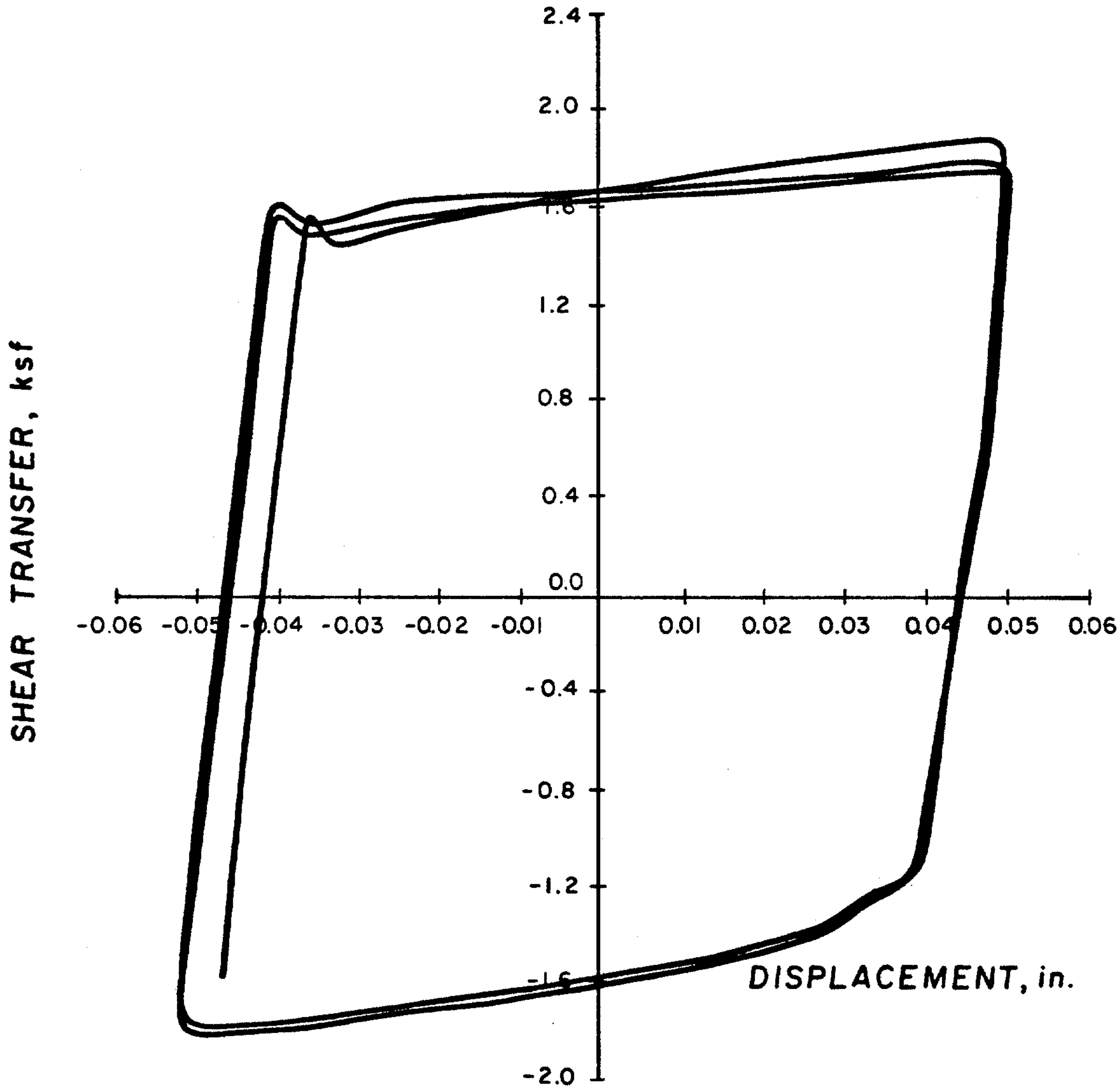
FLUCTUATIONS IN SOIL PRESSURE DURING ONE-WAY CYCLIC TENSION TESTS



RESULTS OF THE TENSION TEST AFTER THE ONE-WAY CYCLIC TENSION TESTS

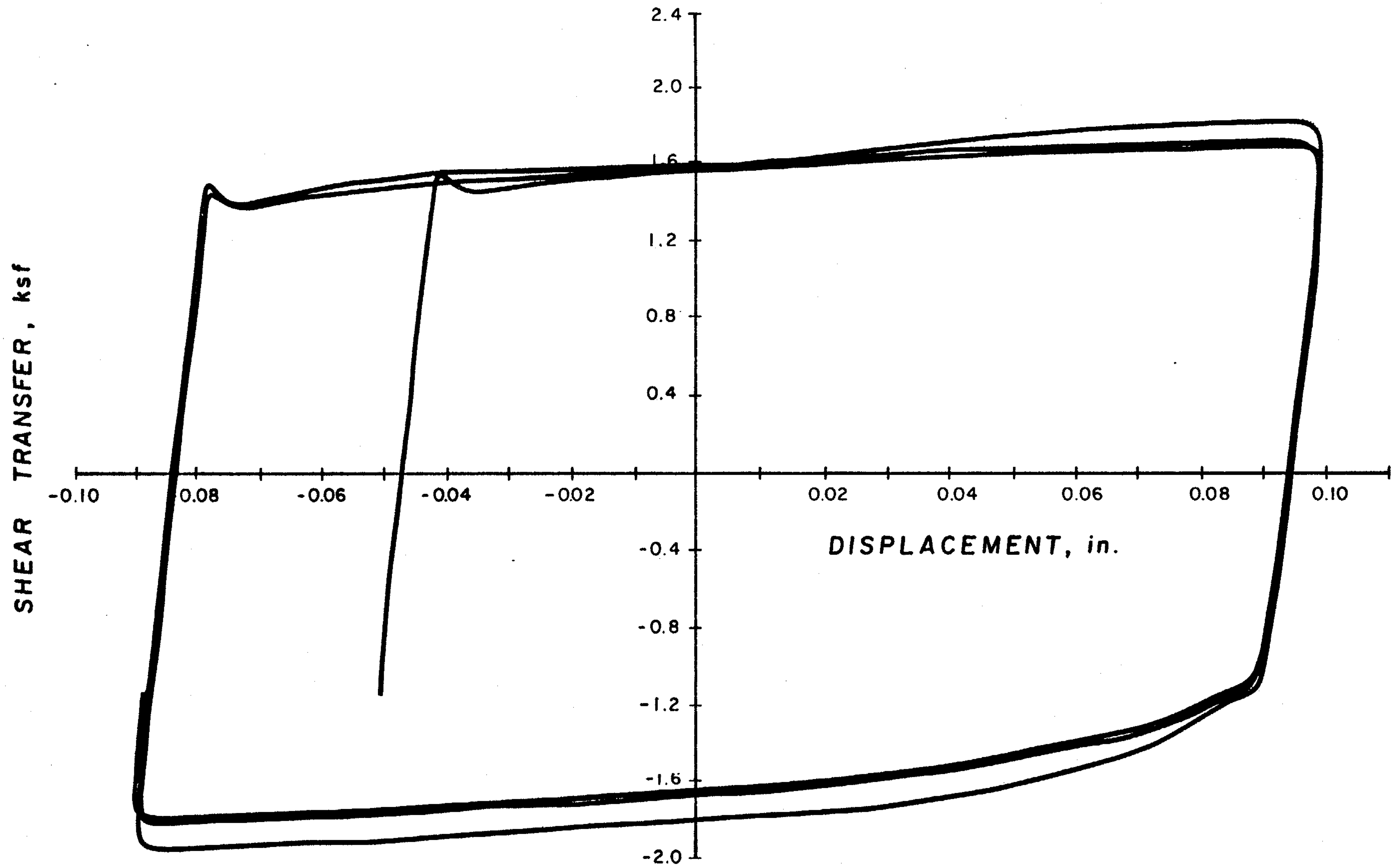


RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST

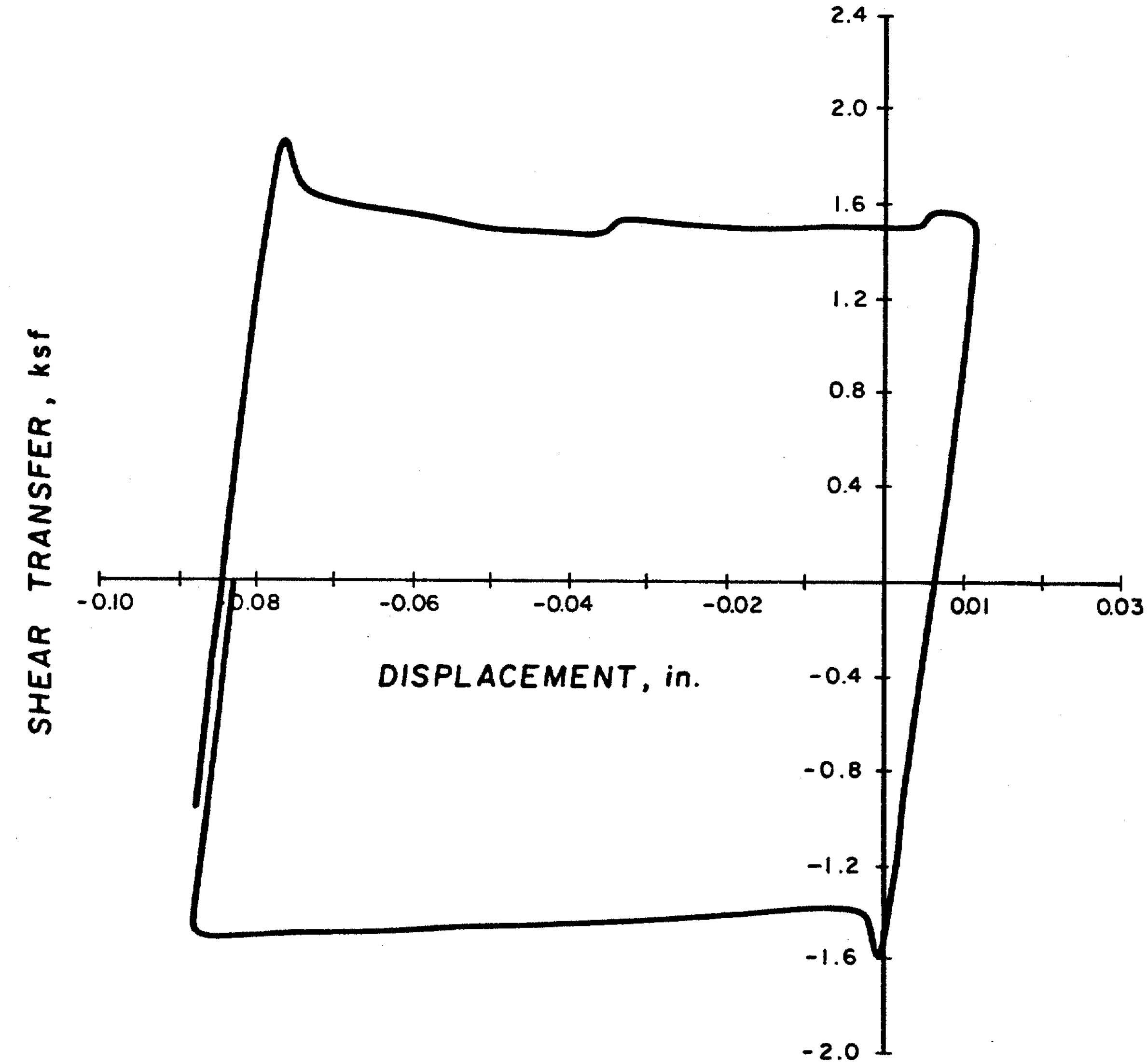


RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST

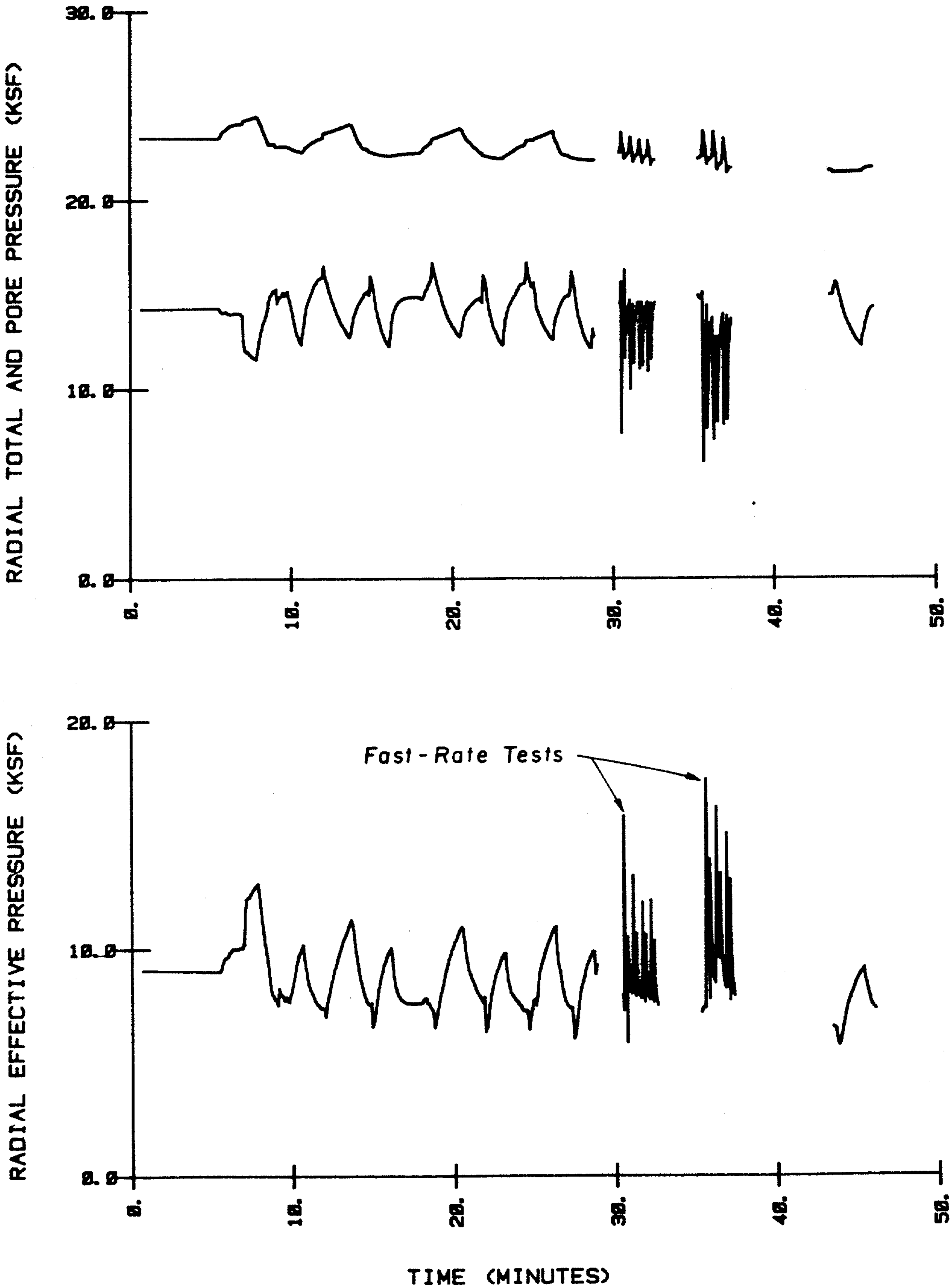
JOB NO. ______



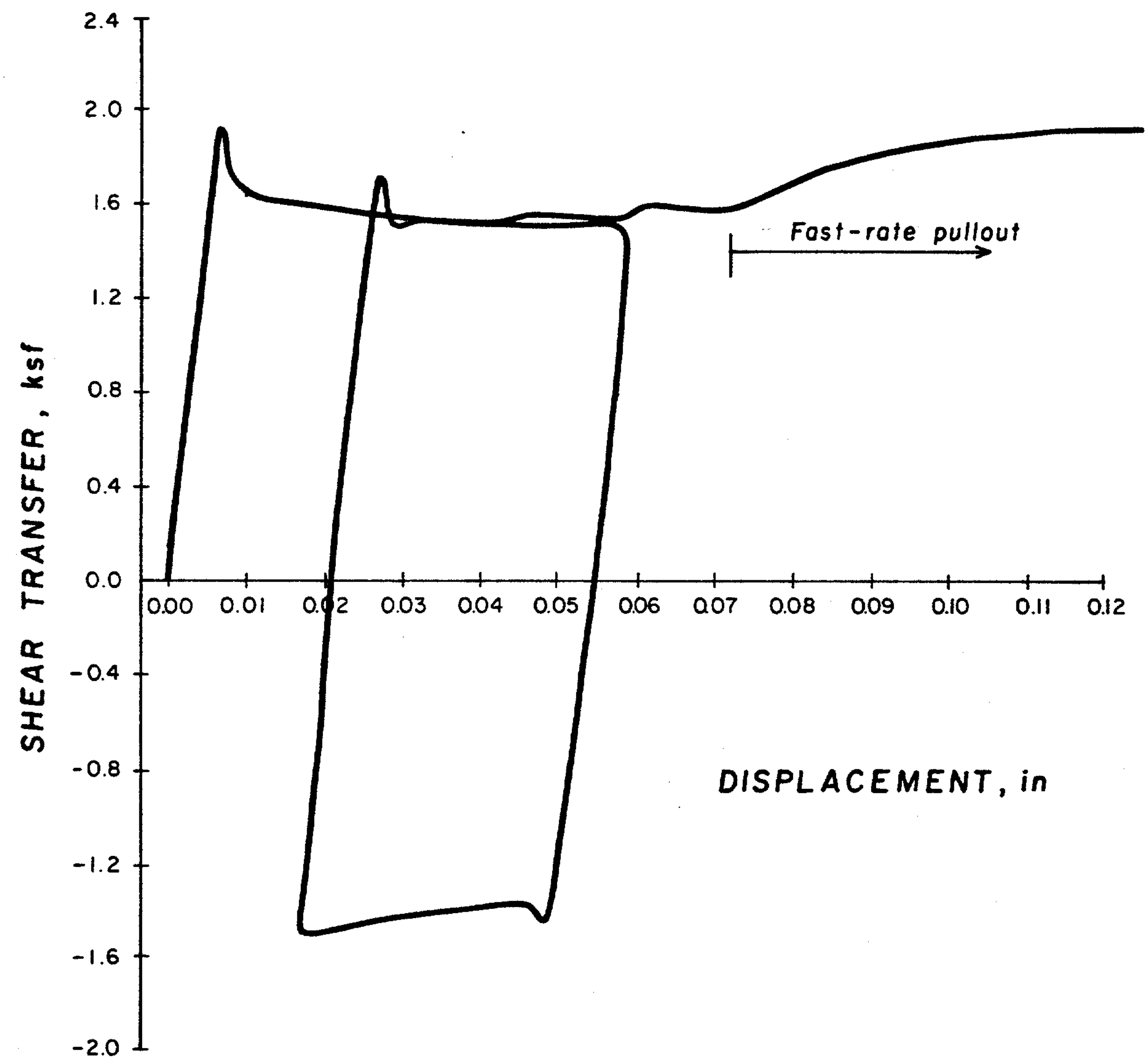
RESULTS OF THE FAST-RATE TWO-WAY CYCLIC TEST WITH DISPLACEMENT LIMITS INCREASED



REPEAT OF THE TWO-WAY CYCLIC TEST



FLUCTUATIONS IN SOIL PRESSURE DURING TWO-WAY CYCLIC LOADING



LOAD TESTS AFTER RECONSOLIDATION, WITH SLIP RATE VARIED

APPENDIX K: CLOSED-END 3-IN. PROBE EXPERIMENT AT THE 210-FT DEPTH

RESULTS OF THE CLOSED-END 3-IN. PROBE EXPERIMENT AT THE 210-FT DEPTH

The experiment with the closed-end 3-in. probe at the 210-ft depth was performed during the period from 1400 hours on 21 April until 1030 hours on 24 April.

The probe was installed by driving, with the impact force applied to the top of the N-rod string by a 300-lb casing hammer. The driving was begun at 14:21 and ended at 14:26.

The variation in soil pressures during consolidation are shown in Plate K-1. As was the case with the X-probe, consolidation was much faster, being almost complete at the time of the immediate test. The maximum radial total pressure recorded after driving was 30.9 ksf; after one minute, the pressure had decreased to 24.2 ksf. The maximum pore pressure was 28.8 ksf; after one minute, the pore pressure had decreased to 22.0 ksf.

The first load test was performed 20 minutes after driving. At this time, the total radial pressure was 19.8 ksf; the pore pressure was 15.9 ksf, indicating a radial effective pressure of 3.9 ksf. The results of the first load test are given in Plate K-2. The shear-displacment behavior displayed a stick-slip response. Such response is partly due to the elastic deformation of the N-rod string. Had the loading system been stiffer, the response would have been normal (i.e., smooth). The maximum shear on the first loading was 0.80 ksf; the first yield was at 0.74 ksf.

Consolidation was allowed to proceed for 65 hours. Although the process was essentially completed in 3 hours, the load test was postponed because of the need to perform additional experiments. During consolidation, the pore pressure stabilized at 14.2 ksf in 5 hours, and remained near 14.2 ksf for the remainder of the process. The total radial pressure decreased to 19.3 ksf during the first hour; then increased, becoming near-constant 17 hours after driving. The radial effective pressure thus also stabilized, at value of 5.8 ksf.

The results of the initial loading to failure in tension after consolidation are shown in Plate K-3. As shown in the plate, the maximum shear during the test was 1.78 ksf, with a value of 1.71 ksf on the first yield. Although the shear transfer was still decreasing when the load was removed, the residual value should be close to the value prior to removing the load, i.e., 1.50 ksf.

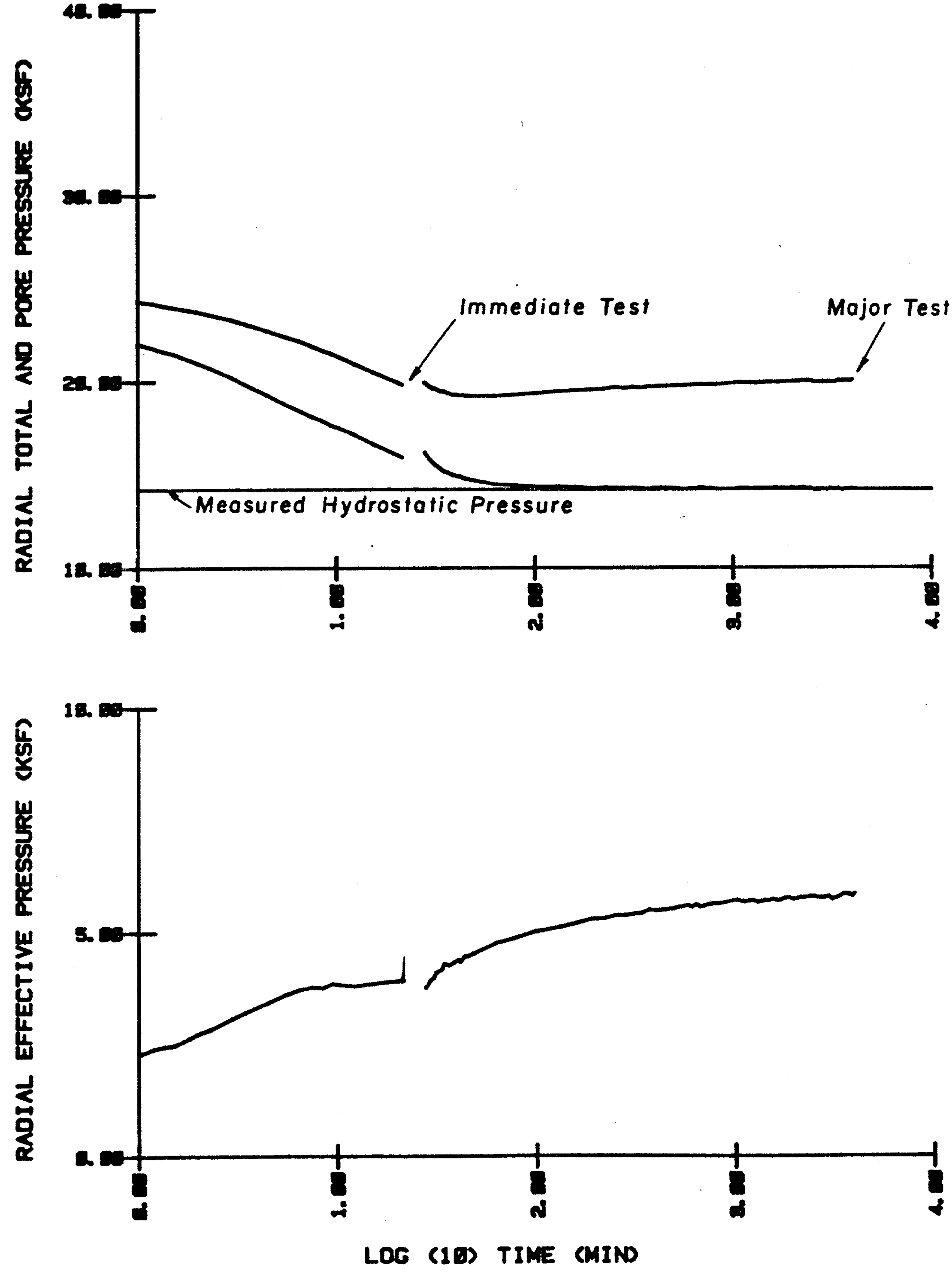
Because of the unusual behavior which was observed, the cyclic tension tests were omitted. The probe was subjected to several cycles of controlled-displacement two-way cyclic loading, with the results shown inPlate K-4. The peak resistance on the first cycle was 1.85 ksf; after three cycles, the shear transfer had been reduced to about 1.3 ksf. As seen in the plate, the shear-displacement response was very irregular.

The rate of loading was then increased, with the results shown in Plate K-5. As shown in the plate, no constant-shear plastic slip was recorded. The maximum shear transfer on the last cycle, just prior to load reversal, was 1.37 ksf.

The loading rate was then reduced, with the results shown in Plate K-6. The behavior was still irregular, with a value of shear transfer near the point of load reversal being 1.17 ksf.

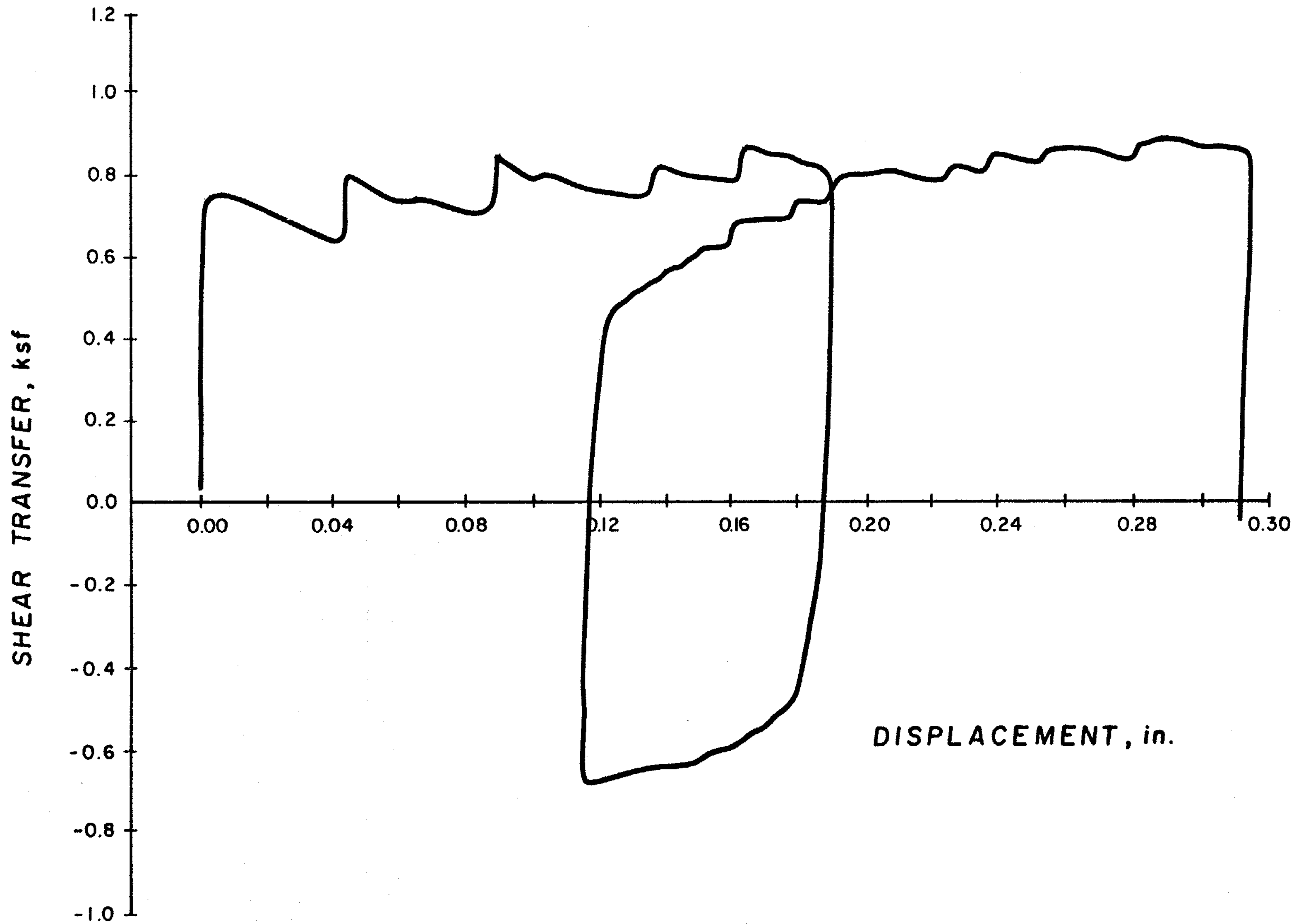
The fluctuations in soil pressure during the two-way cyclic tests are given in Plate K-7. As noted in the X-probe experiment, the fluctuations in the soil pressures are larger in magnitude than those observed in the clays, but with the same pattern of response.

The probe was then removed from the boring, with one final loading to failure in tension. After yield, the rate of loading was increased, with the results shown in Plate K-8. On the second yield plateau, the shear transfer was 1.03 ksf; the corresponding slip rate was 0.00168 in./sec. At the slip rate of 0.01765 in./sec, the maximum value of shear transfer reached 1.80 ksf. This corresponds to a rate effect of 73 percent per log cycle.



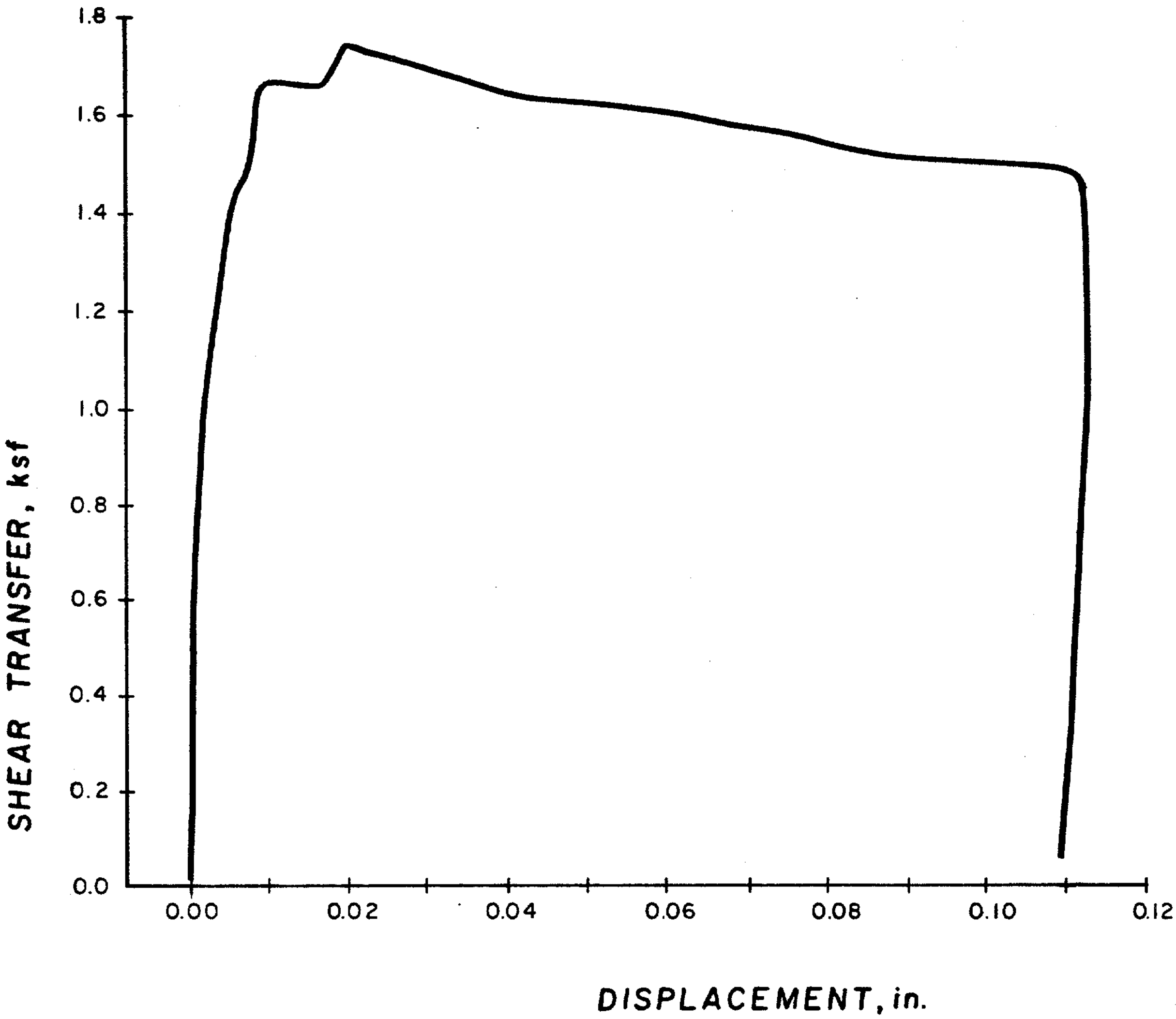
VARIATIONS IN SOIL PRESSURES DURING CONSOLIDATION

JOB NO.

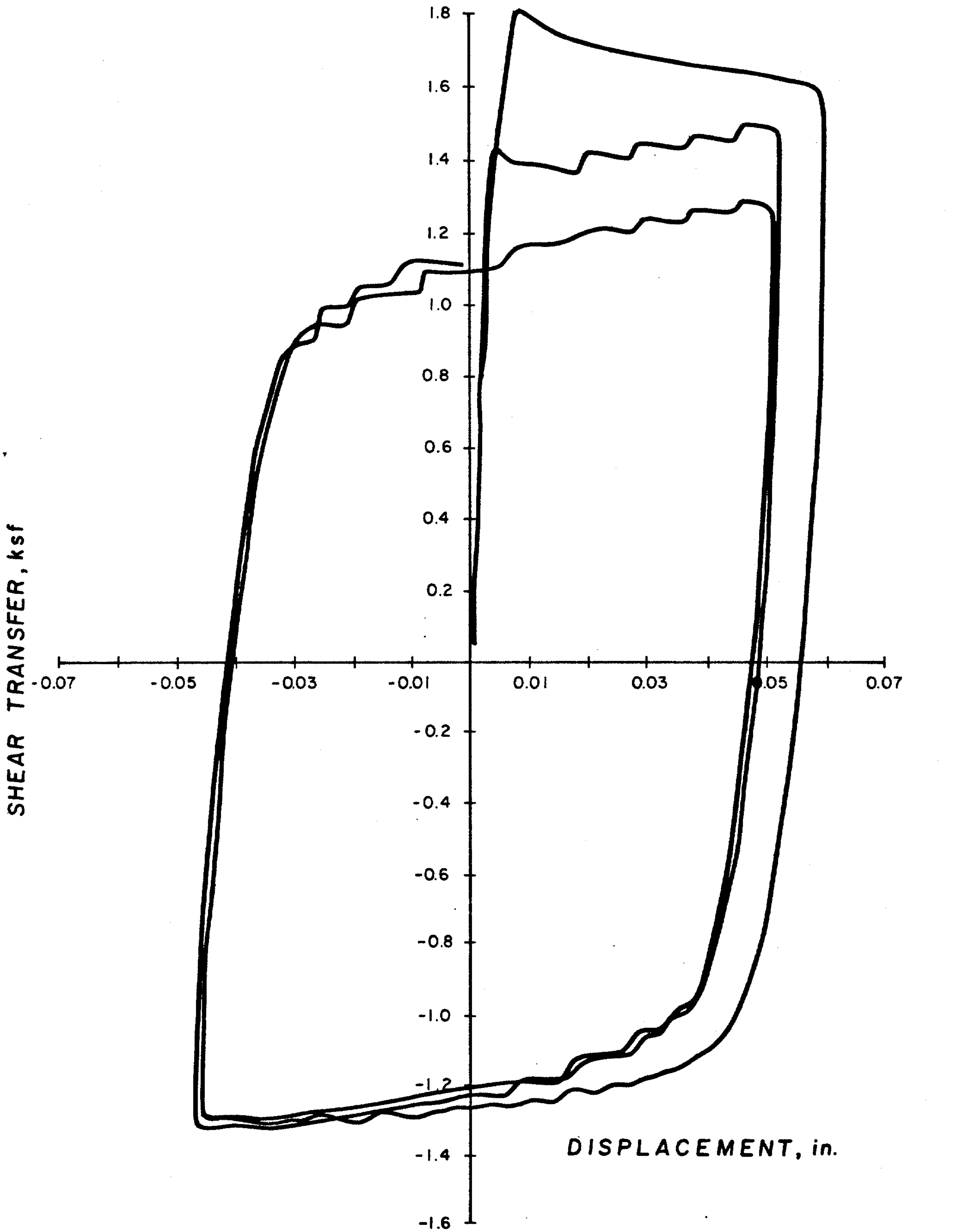


RESULTS OF THE LOAD TEST IMMEDIATELY AFTER INSTALLATION

The Earth Technology Corporation

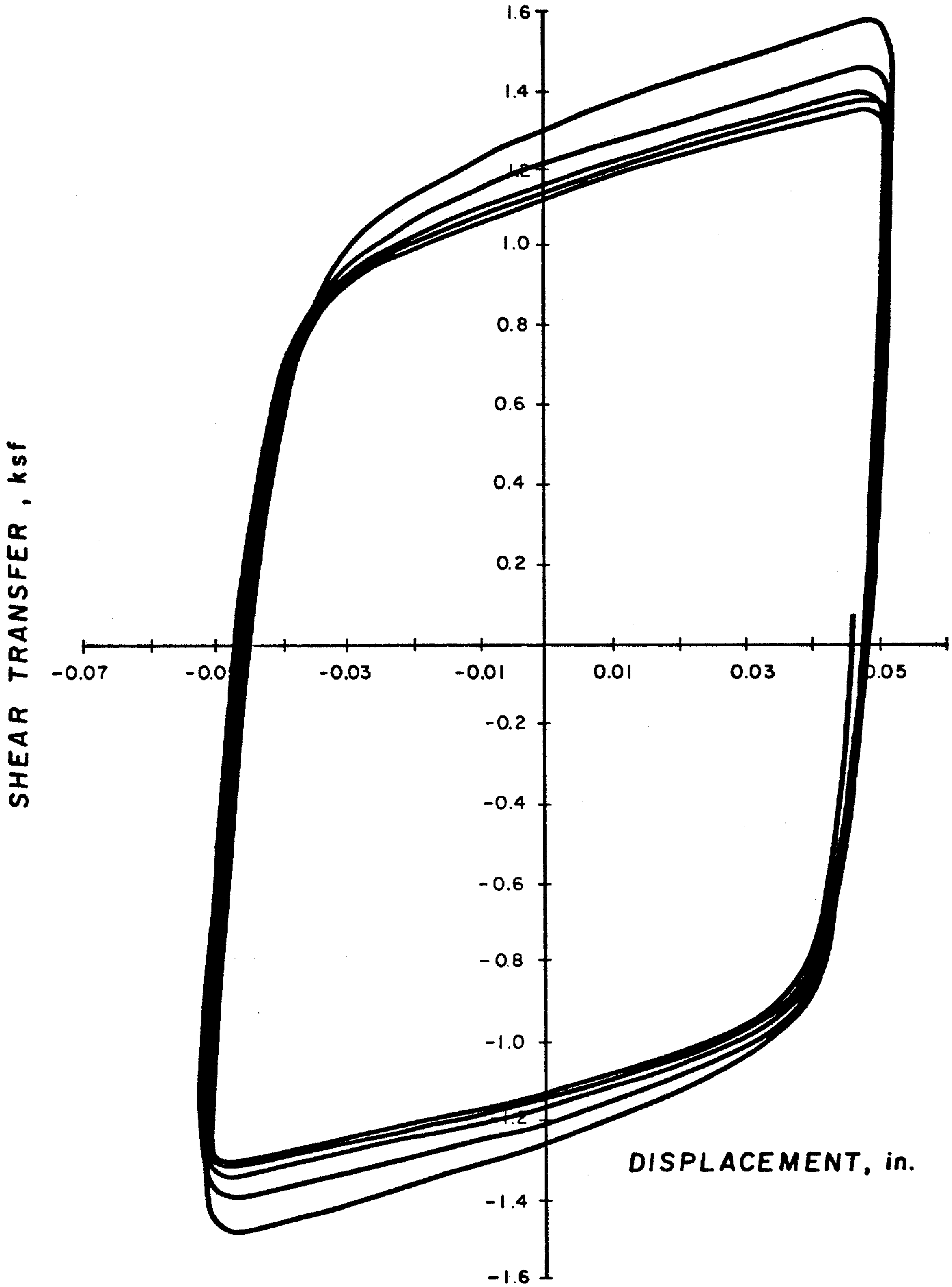


RESULTS OF THE INITIAL LOADING IN TENSION AFTER CONSOLIDATION

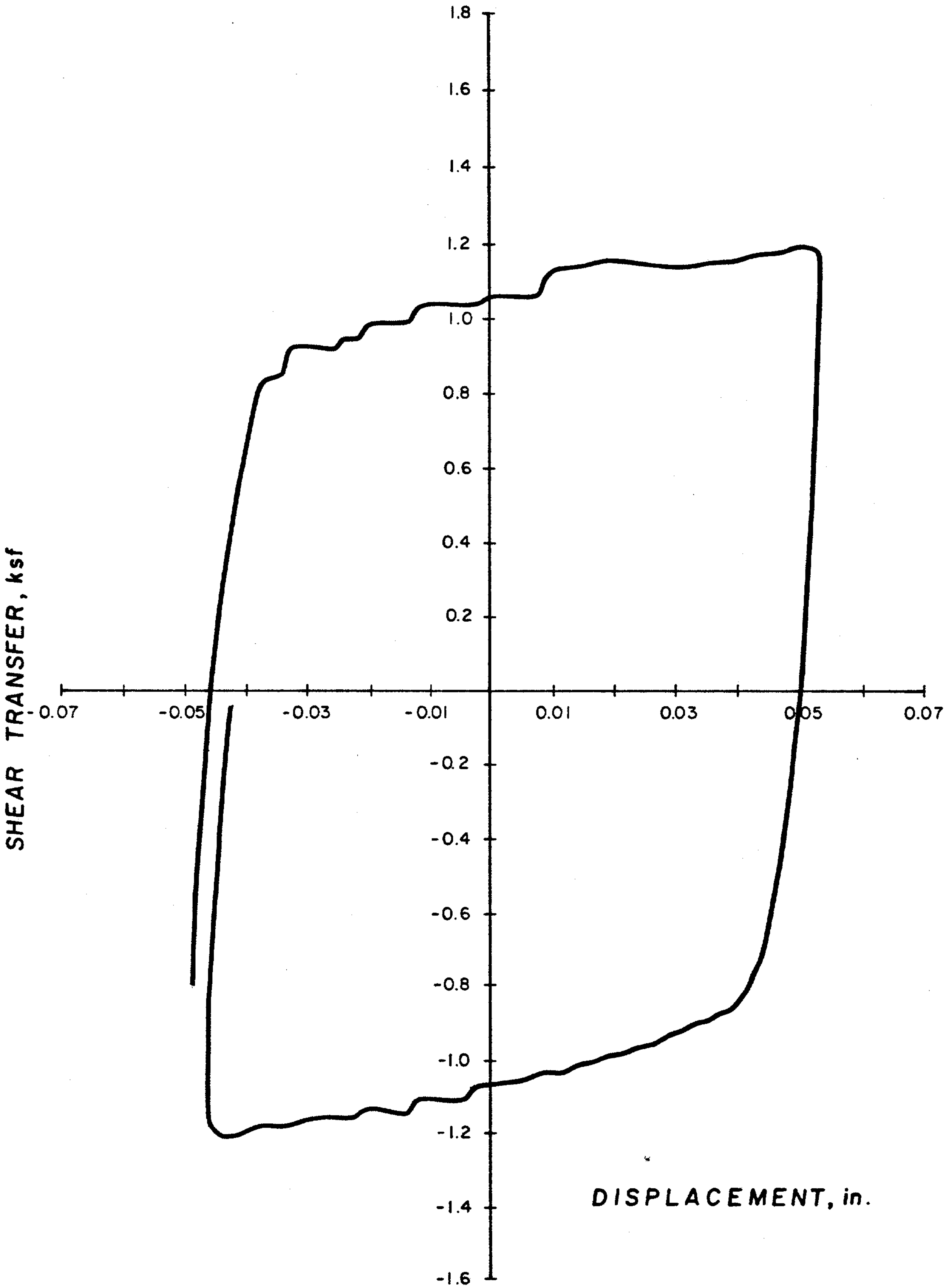


RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST

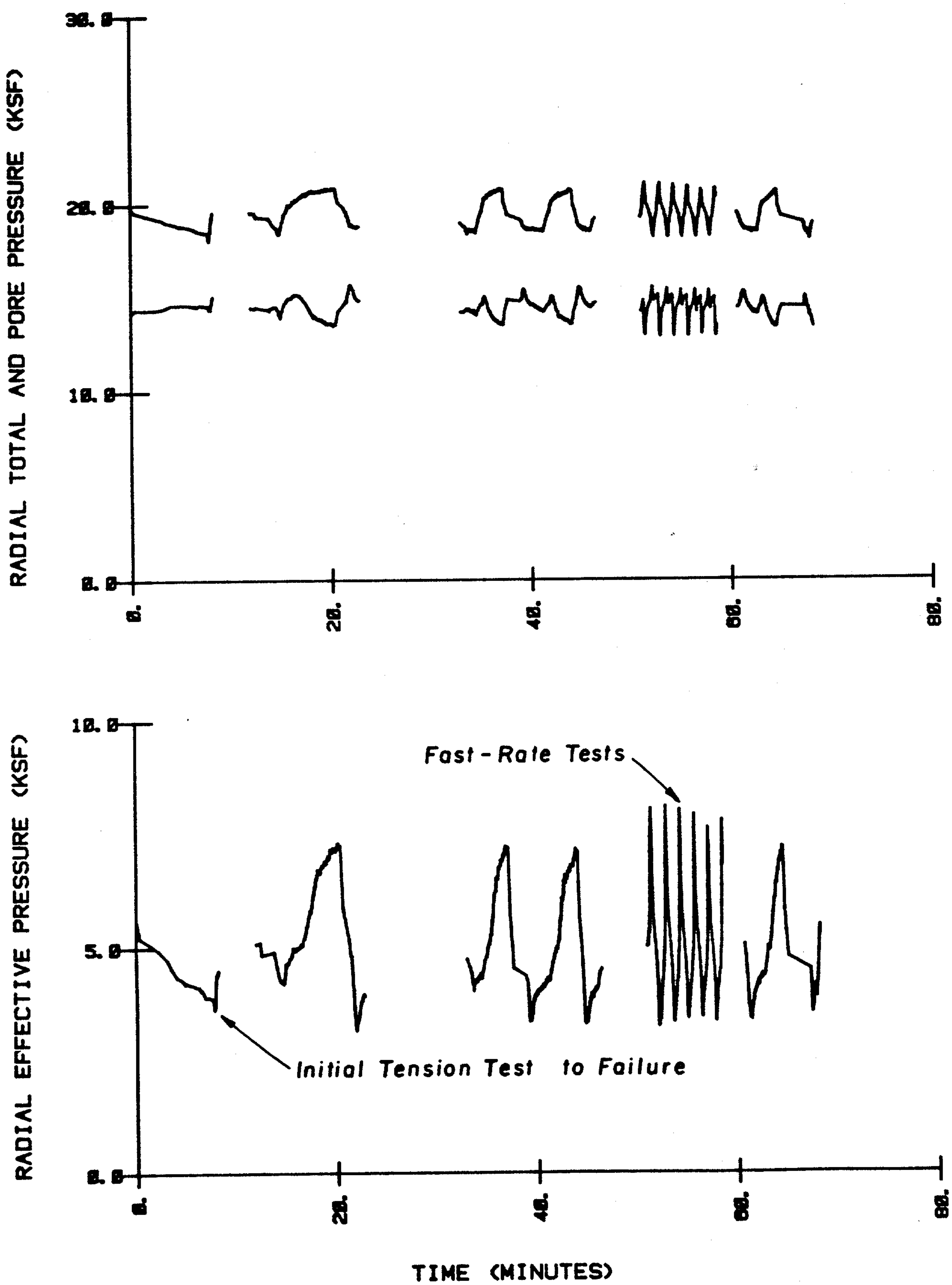
The Earth Technology Corporation



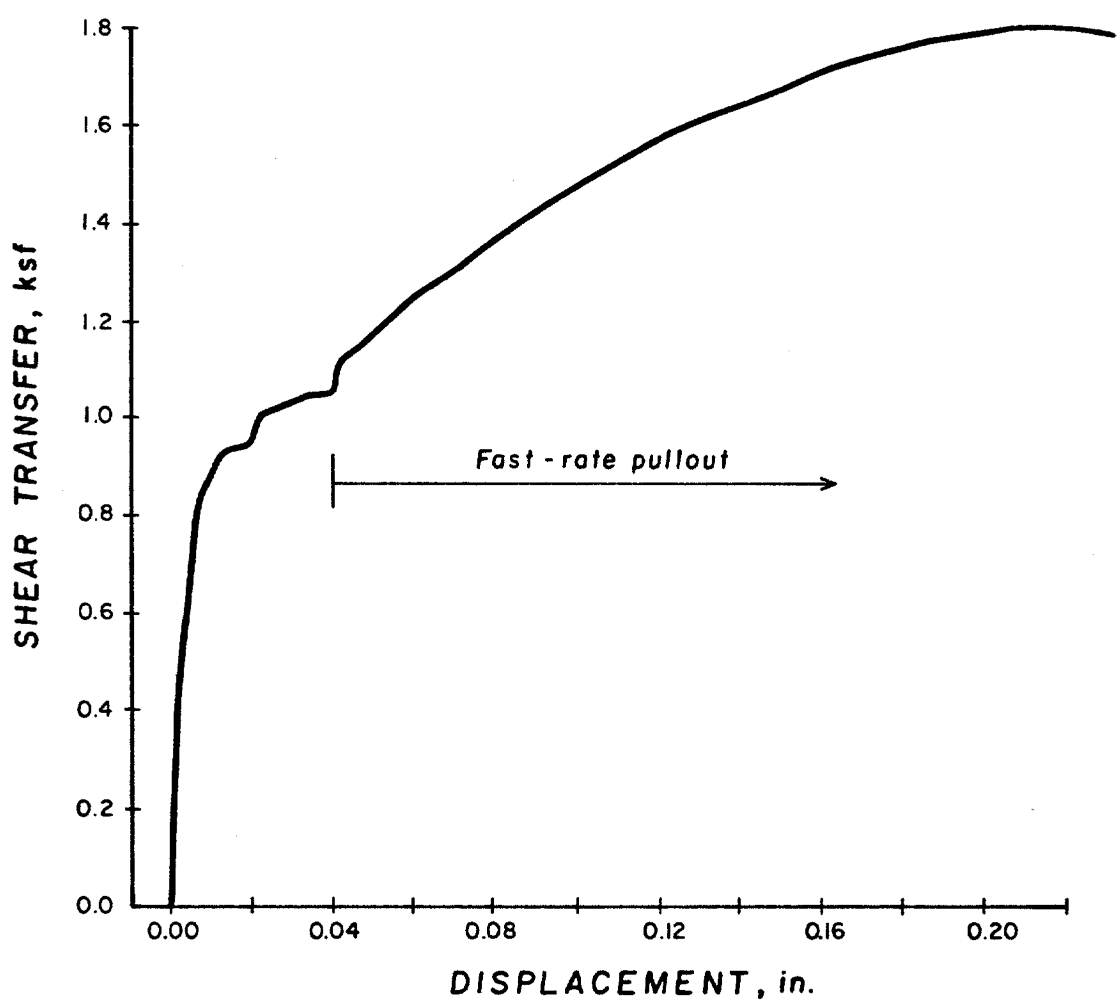
RESULTS OF THE FAST RATE TWO-WAY CYCLIC TEST



RESULTS OF REPEAT TWO-WAY CYCLIC TEST



FLUCTUATIONS OF SOIL PRESSURES DURING TWO-WAY CYCLIC TESTS



RESULTS OF THE LOAD TEST IN TENSION
PEFORMED AFTER ADDITIONAL CONSOLIDATION

APPENDIX L: OPEN-END 3-IN. PROBE EXPERIMENT AT THE 210-FT DEPTH

RESULTS OF THE OPEN-END 3-IN. PROBE EXPERIMENT AT THE 210-FT DEPTH

The experiment with the open-end 3-in. probe at the 210-ft depth was performed during the period from 1100 hours on 16 April until 0800 hours on 20 April.

The probe was installed by driving. The impact forces were applied to the top of the N-rod string by a 300-lb casing hammer. The driving was begun at 10:50, and was completed at 11:04.

The variation in the soil pressures during consolidation are shown in Plate L-1. The rapid consolidation and low values of effective pressure were not anticipated; however, the experiment was performed according to the schedule, which had been made assuming plastic soils at this depth. Because of the need to perform several other experiments during the same time period, the testing time was not changed, although the consolidation had proceeded to completion.

The maximum radial total pressure which was measured after driving was 23.0 ksf. The maximum value of pore pressure was 21.5 ksf. At the time of the first load test, 20 minutes after driving, the radial total pressure had decreased to 19.7 ksf. The corresponding pore pressure was 16.7 ksf. Thus, the radial effective pressure increased from 1.5 to 3.0 ksf.

The results of the first load test are given in Plate L-2. The initial peak shear transfer was 0.12 sf. After the second failure in tension, the shear transfer continued to increase, being 0.16 ksf at the end of the test.

Following the end of the test, the radial total pressure decreased from 17.8 ksf to 17.6 ksf, and remained reasonably constant thereafter. The pore pressure decreased to 14.4 ksf in 3 hours, and also remained reasonably constant. As seen in the Plate L-1, the value of radial effective pressure also remained a constant 3.2 ksf.

The load test to failure in tension after a period of 73 hours is shown in Plate L-3. The peak value of shear transfer was 0.95 ksf, or about half the value measured during the tests with the full-displacement probes. The shear transfer decreased markedly after yield; being only 0.65 ksf prior to load reversal.

At the end of the load test, the radial total pressure was 17.1 ksf, with a pore pressure of 15.5 ksf. After the load test, the pore pressures equilibrated in 28 minutes, as did the radial total pressures. The next series of tests were begun after 41 minutes. At the end of this waiting period, the radial total pressure had reached 16.9 ksf, and the pore pressure 14.4 ksf.

The probe was then subjected to a series of one-way cyclic tension tests, with the results summarized in Plate L-4. As shown in the plate, the accumulation of permanent upward displacement did not begin until the load level was increased to a value equal to 80 percent of the initial maximum shear transfer and accelerated accumulation, with rapid pullout, occurred only at a load level of 107 percent of the initial maximum shear transfer.

The fluctuations in the soil pressures which accompanied the one-way cyclic tension tests are given in Plate L-5. The pressure fluctuations are small, especially when compared to the comparable test series performed on the X-probe at this depth.

The probe was then subjected to a slow monotonic loading to failure, followed by three cycles of controlled-displacement loading. The results of the two-way cyclic tests are shown in Plate L-6. As shown in the plate, a significant decrease in the shear transfer accompanied the first reversal, with the shear transfer rapidly stabilizing. The limiting shear transfer on the first loading was 0.97 ksf; after reversal, the shear transfer stabilized at 0.63 ksf.

The rate of loading was then increased, with the results shown in Plate L-7. The limiting shear transfer on the third cycle was 0.80 ksf, at a rate of slip of 0.0093 in./sec.

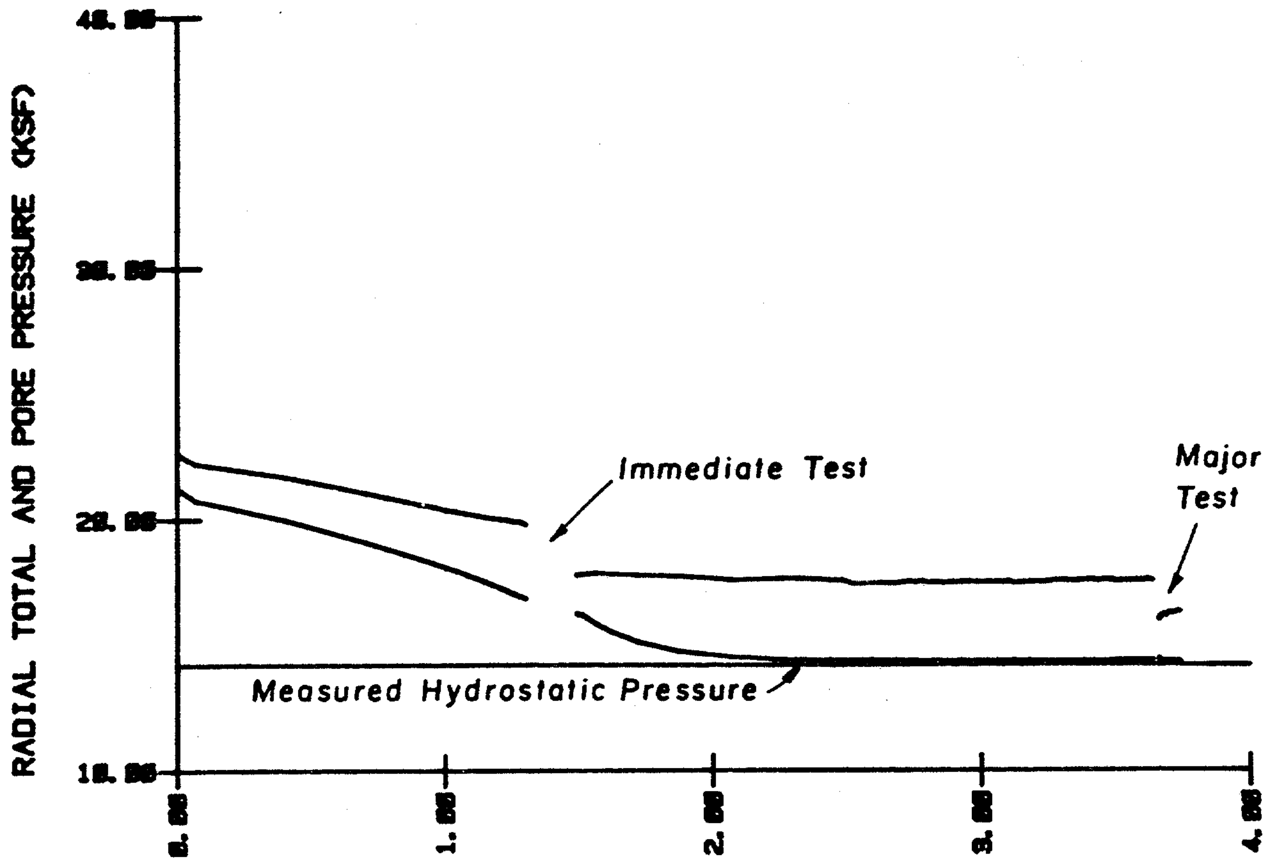
The rate of slip was then decreased to 0.00055 in./sec, with the resulting hysteresis loop shown in Plate L-8. The limiting shear was decreased to 0.67 ksf at the slower rate, for an indicated increase of 16 percent per log cycle of increase in slip rate.

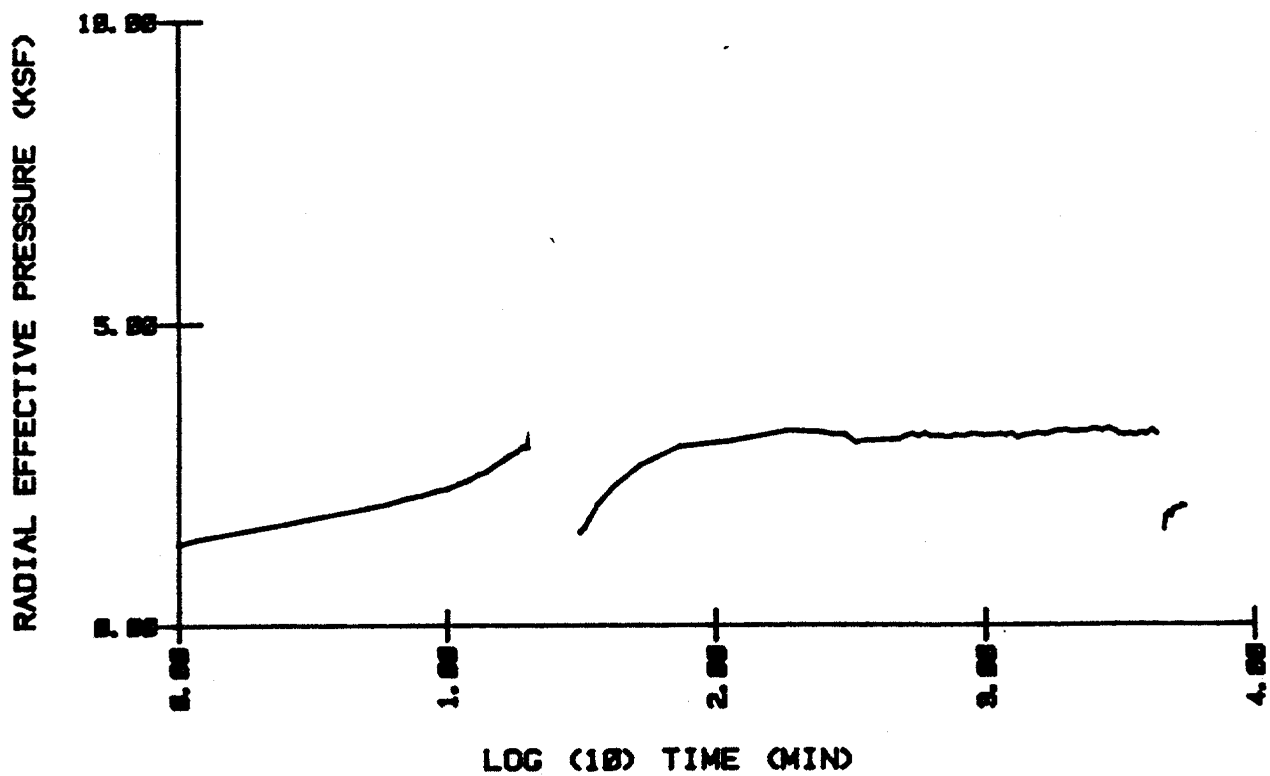
The fluctuations in soil pressures which accompanied the two-way cyclic loading are shown in Plate L-9. The fluctuations follow the same pattern as before, but are, again, smaller in magnitude than those observed in the clay.

The probe was then allowed to rest for 15 1/2 hours, during which time the soil pressures changed only slightly . The pore pressure decreased 0.1 ksf, to a value of 14.4. ksf, while the radial total and effective pressures increased by 0.25 and 0.35 ksf, respectively.

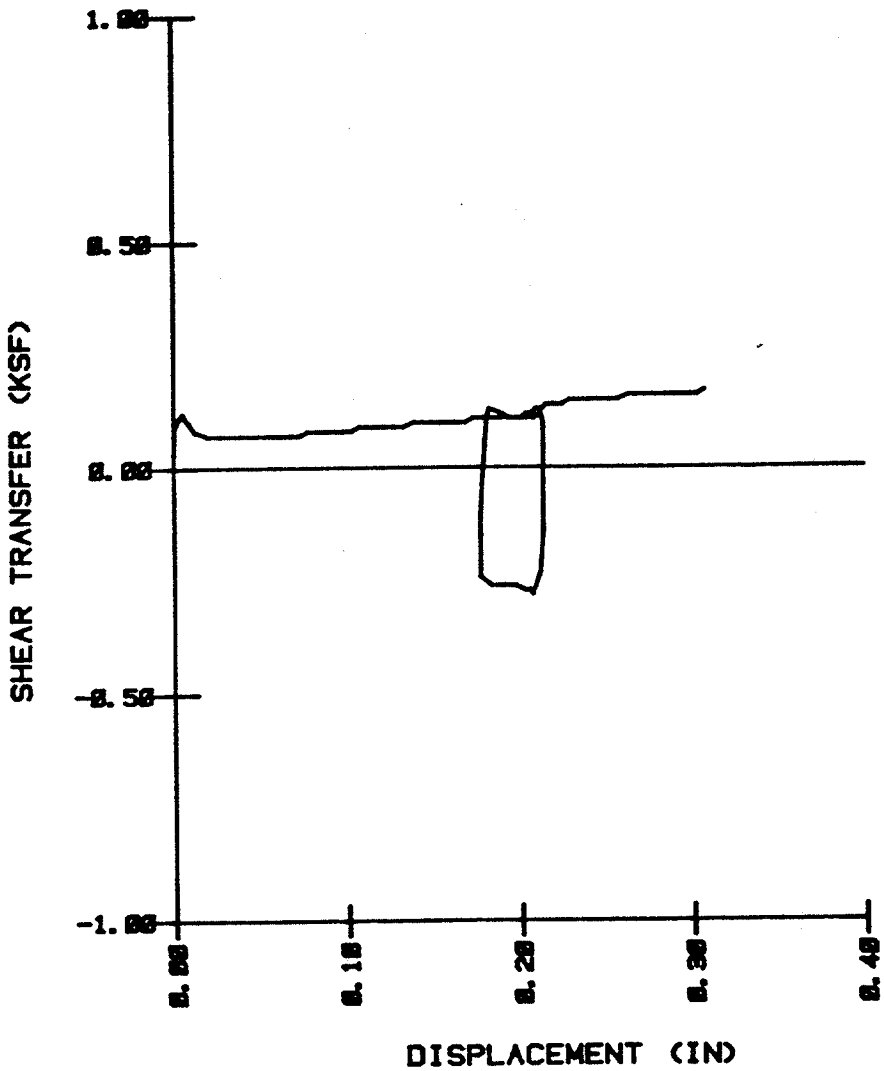
The probe was then loaded to failure in tension, with the rate again varied during plastic slip. The results are shown in Plate L-10, which shows a value of peak shear transfer of 0.82 ksf and a residual shear of 0.72 ksf with a slip rate of 0.00064 in./sec. The rate was then increased to 0.0179 in./sec, with an increase in the shear transfer to 1.02 ksf, indicating an increase of 29 percent per log cycle of increase in slip rate.

Following this test, the probe was removed from the boring.

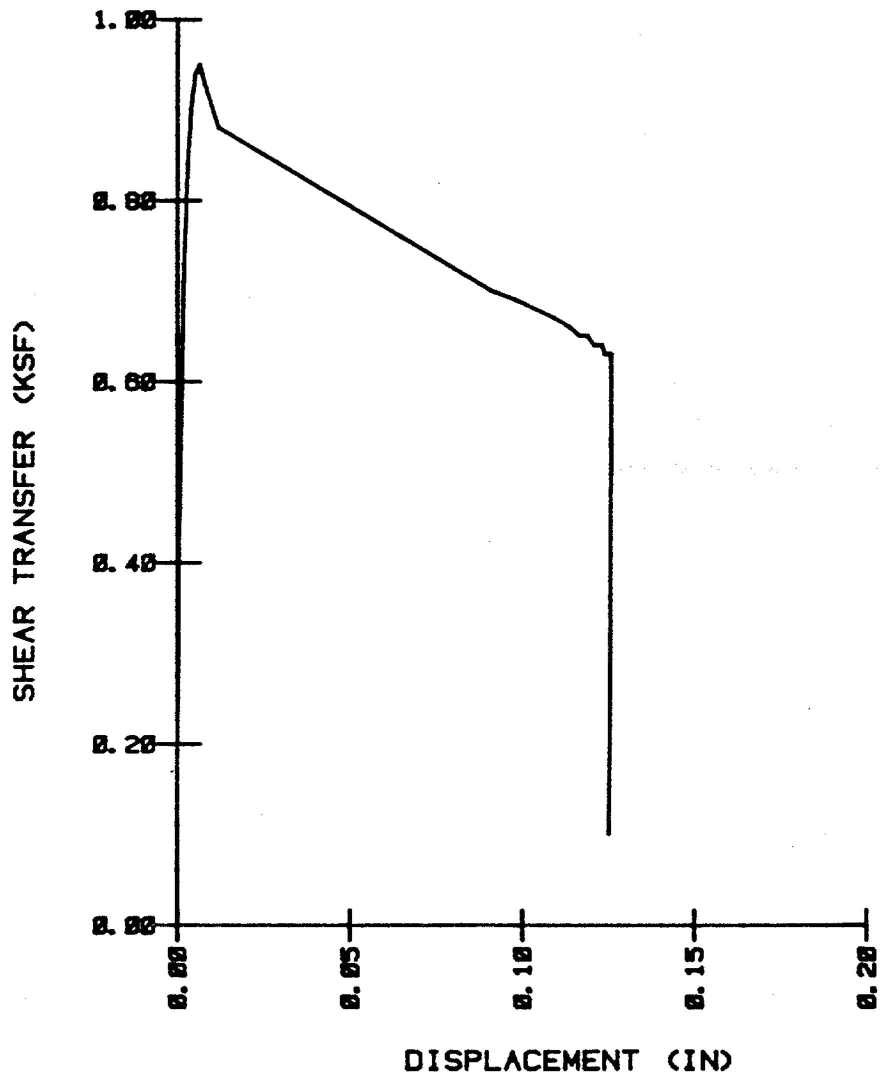




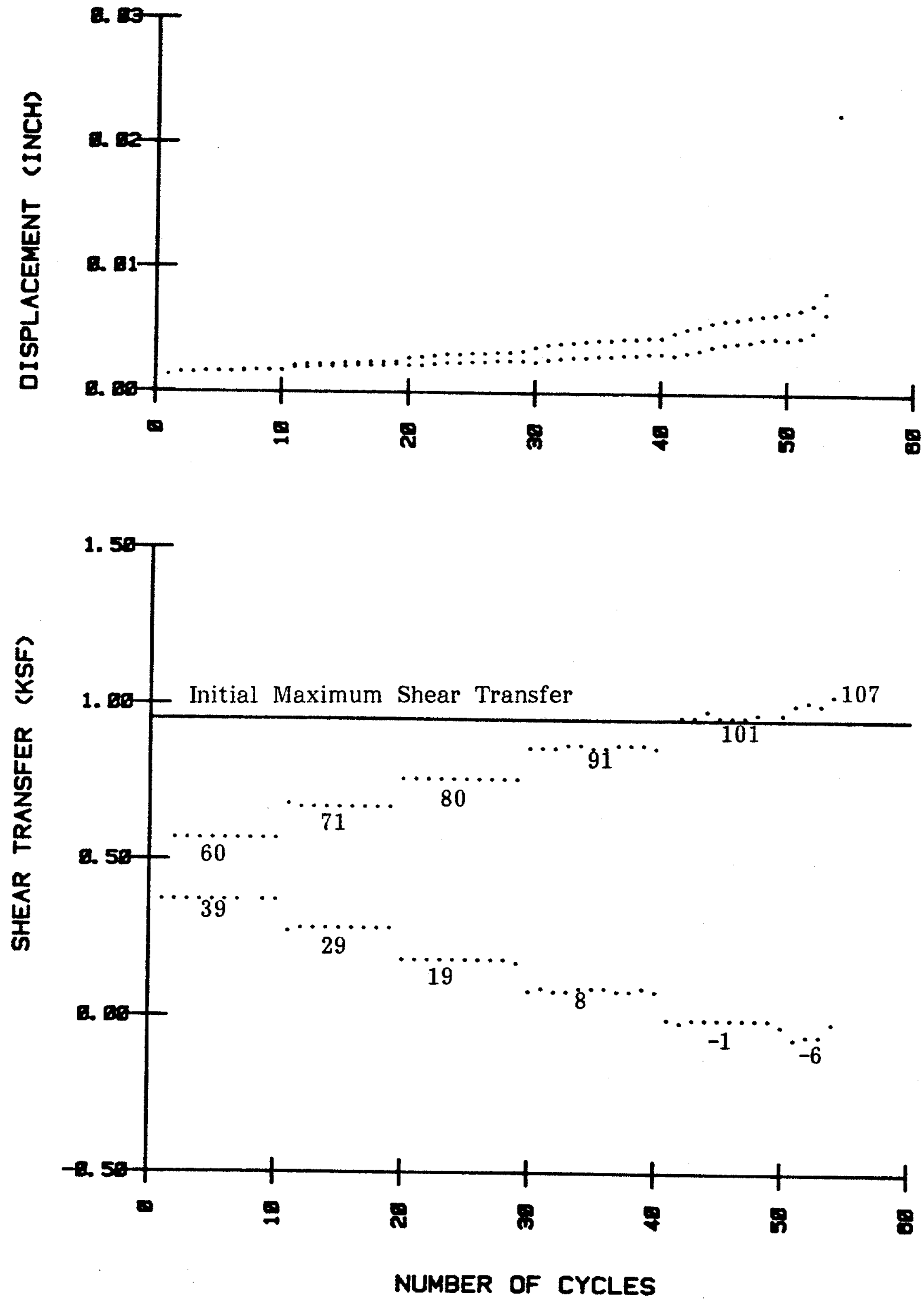
VARIATIONS OF SOIL PRESSURES DURING CONSOLIDATION



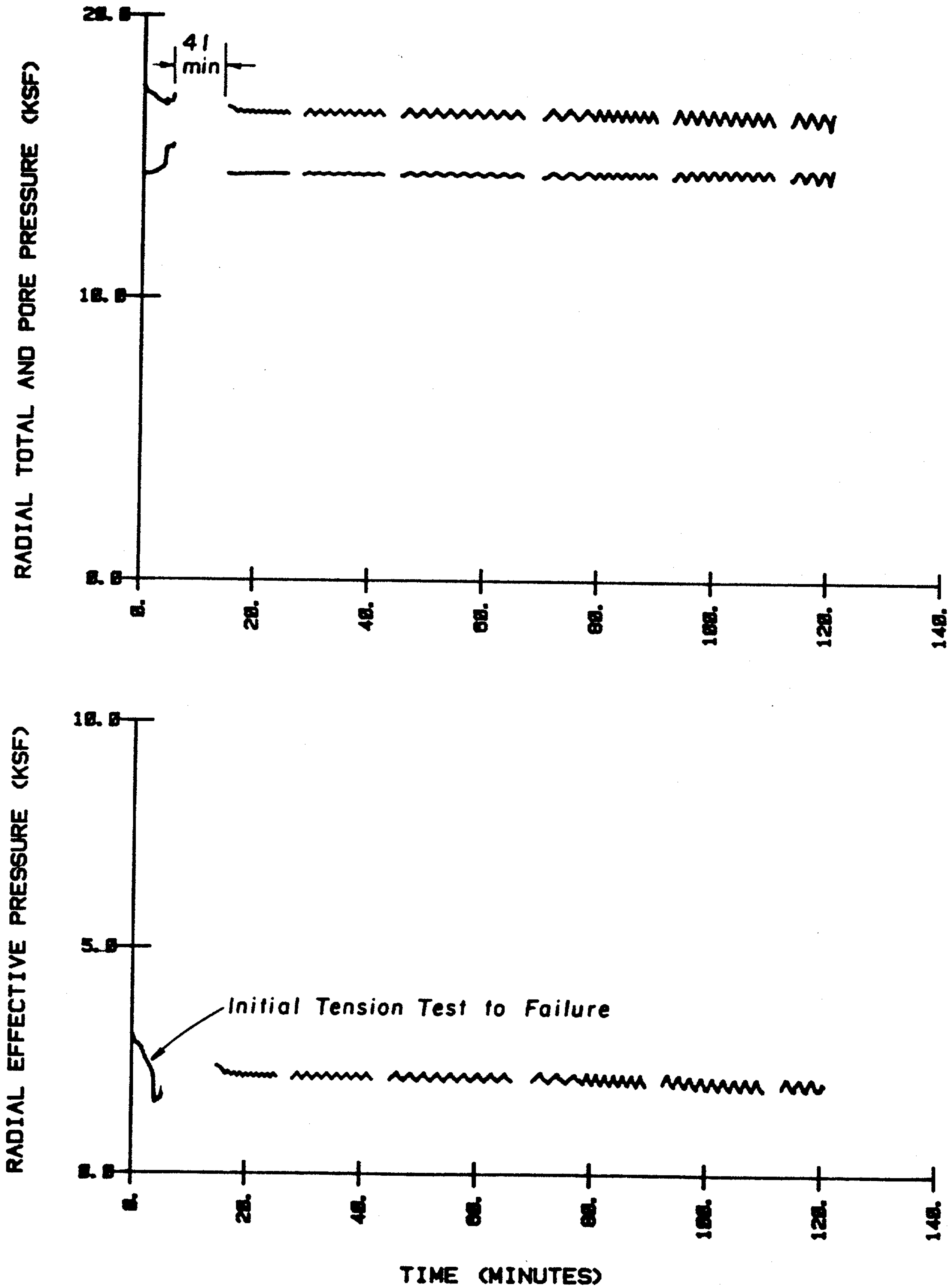
RESULTS OF IMMEDIATE LOAD TEST



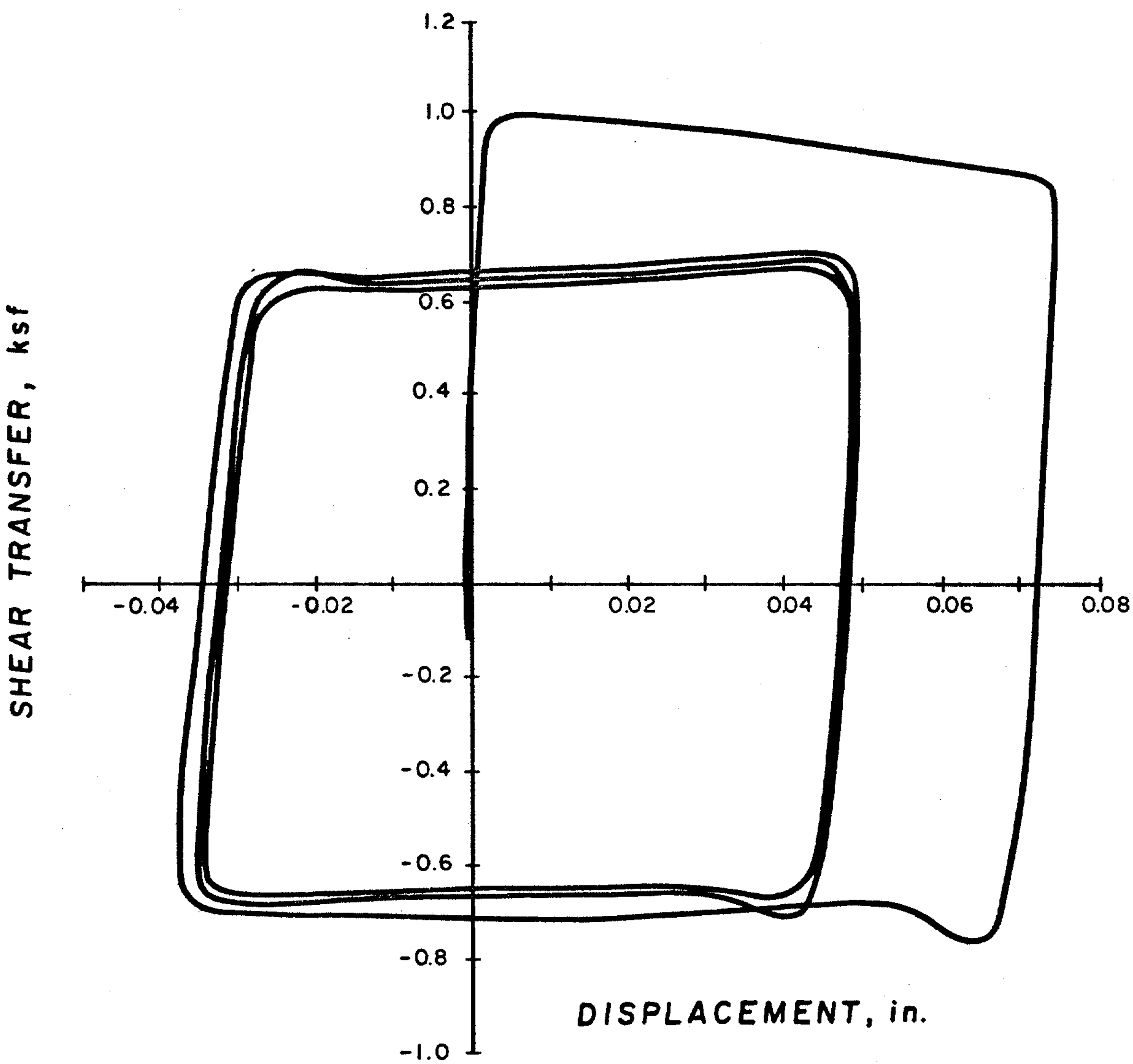
RESULTS OF THE INITIAL LOAD TEST AFTER CONSOLIDATION



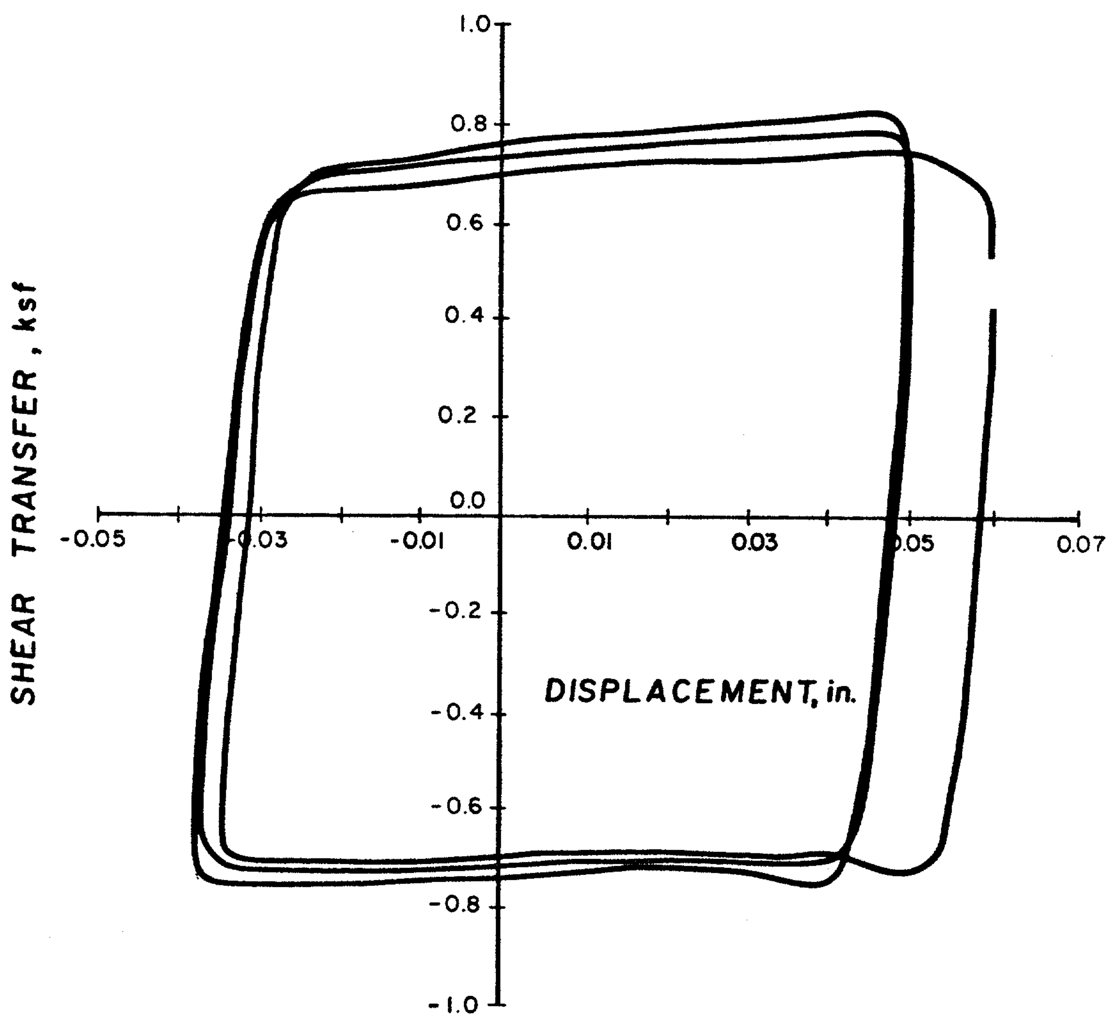
RESULTS OF THE ONE-WAY CYCLIC TENSION TESTS



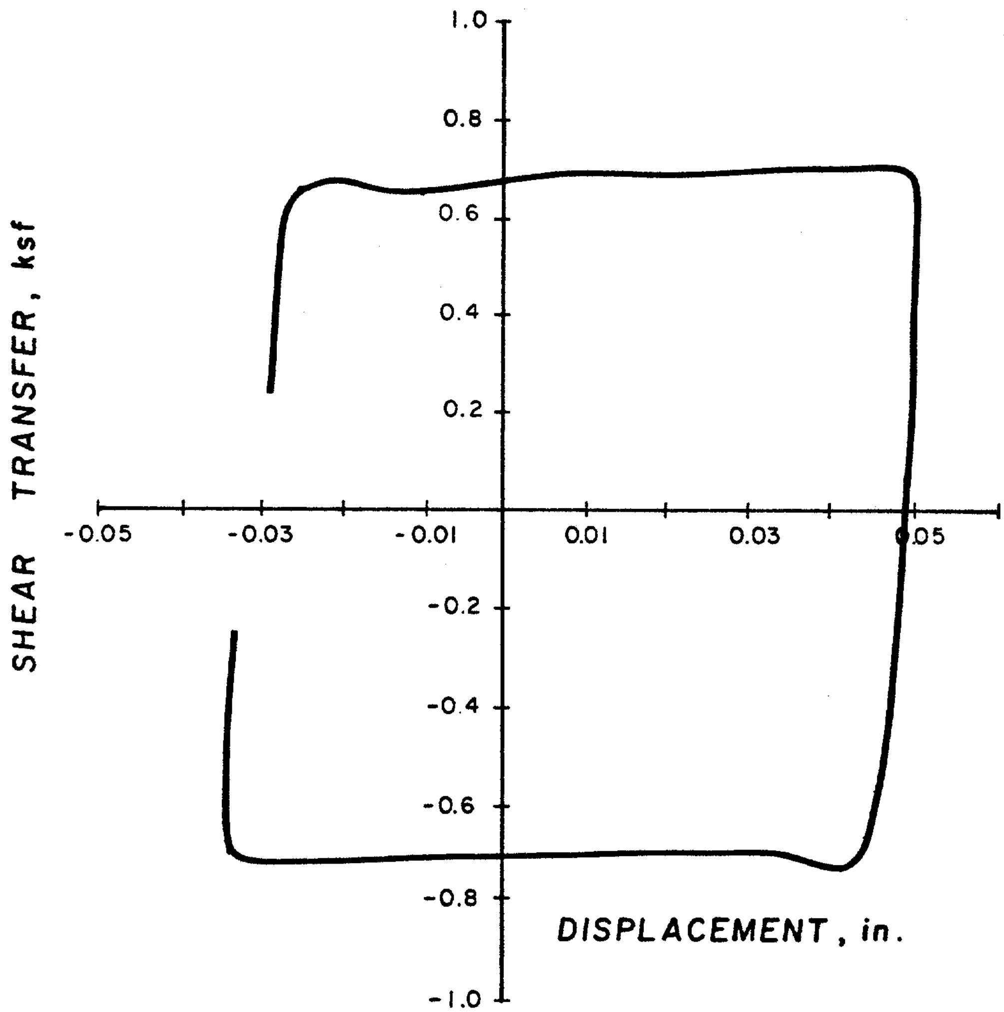
VARIATION OF SOIL PRESSURES DURING ONE-WAY CYCLIC TENSION TESTS



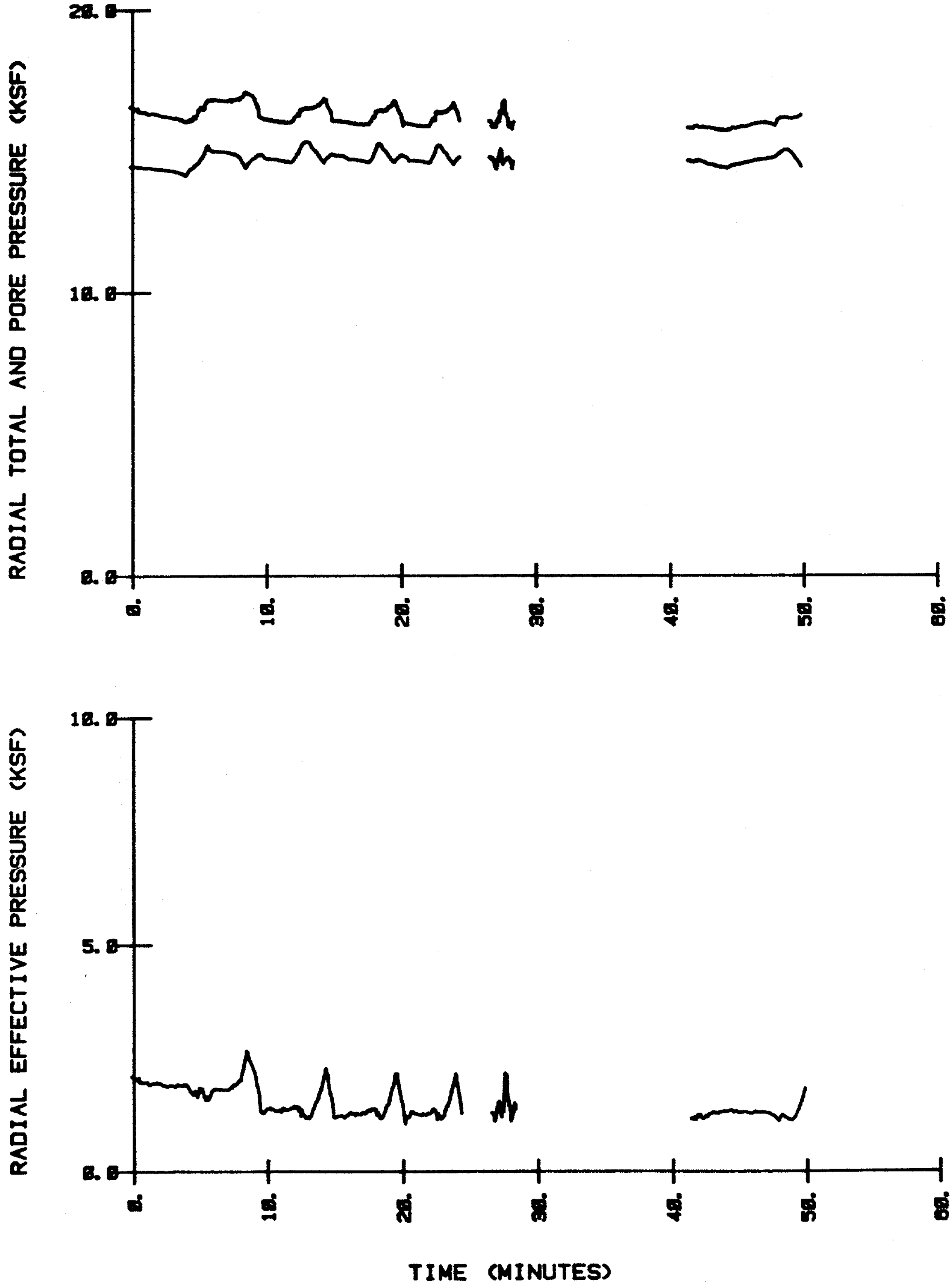
RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST



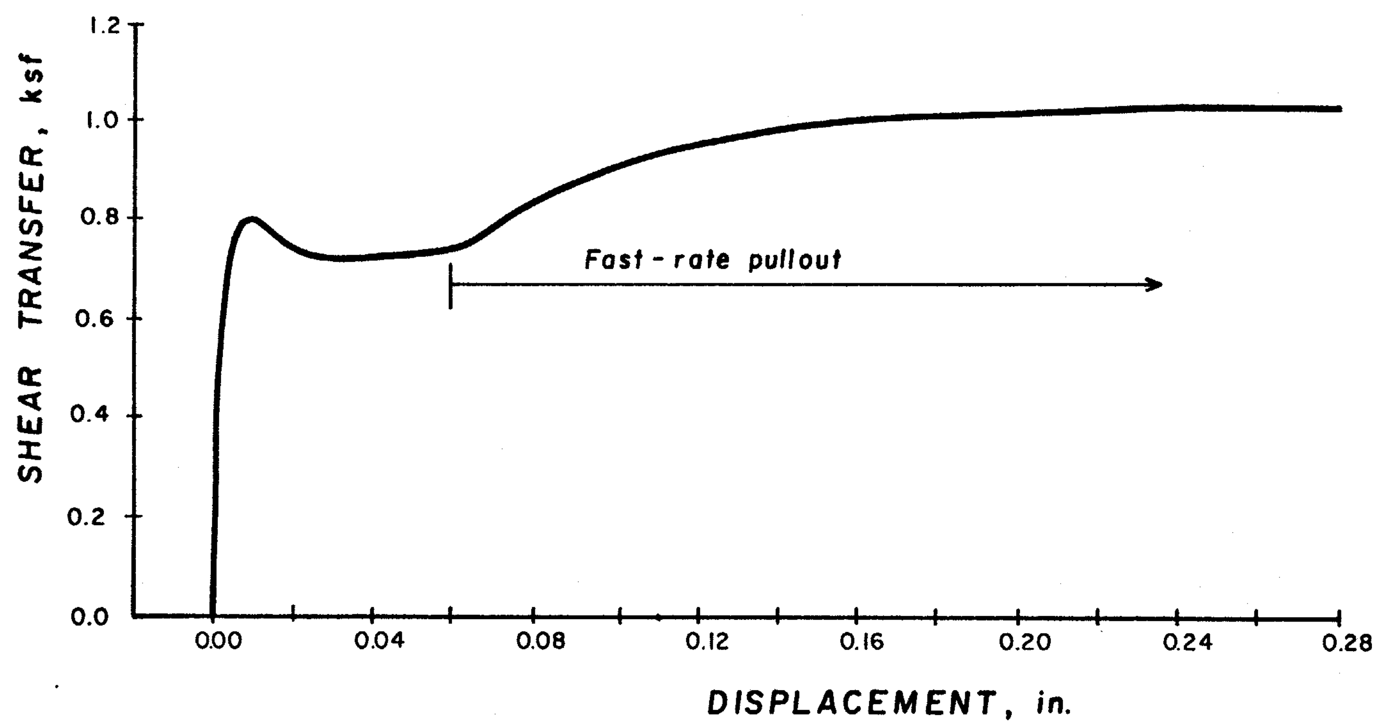
RESULTS OF THE FAST RATE TWO-WAY CYCLIC TEST



RESULTS OF THE REPEAT TWO-WAY CYCLIC TEST



FLUCTUATIONS IN SOIL PRESSURES DURING
TWO-WAY CYCLIC LOADING



RESULTS OF THE VARIABLE-RATE TEST AFTER ADDITIONAL CONSOLIDATION

APPENDIX M: X-PROBE EXPERIMENT AT THE 229.5-FT DEPTH

RESULTS OF THE X-PROBE EXPERIMENT AT THE 229.5-FT DEPTH

The X-probe experiment at the 229.5-ft depth was performed during the period from 1430 hours on 23 April until 0800 hours on 24 April, 1984.

The probe was installed using the drawdown on the drilling rig to push the probe into the soil. During the push, the probe encountered several soil layers of differing stiffness, which were indicated by considerable variations in the force required to advance the probe. At a penetration of 6 in. above the intended penetration depth of 230-ft, the probe tip encountered a very dense layer of soil which could not be penetrated by the drilling rig drawdown. It was therefore decided to stop the push, and to test the probe at the shallower penetration.

The variations in the soil pressures during consolidation are shown in Plate M-1. Values of pore pressure that exceeded 33.8 ksf are not shown. The range of the amplifier had been preset so that the voltage corresponding to 33.8 ksf was the maximum output voltage. At higher levels of input voltage, the output remained constant, therefore, the maximum pore pressure was not recorded.

The maximum total pressure which was recorded after the push was 39.2 ksf. Ten minutes after installation, the recorded pore pressure dropped below 33.8 ksf, at which time the total radial pressure was 37.2 ksf. The first load test was begun 23 minutes after installation. At this time, the total radial pressure had decreased to 36.0 ksf and the pore pressure to 33.2 ksf, indicating a value of radial effective pressure of 2.8 ksf.

A portion of the results of the first load test are given in Plate M-2. The plate contains the shear-displacement behavior which was recorded after the first reversal. The shear transfer varied from 1.65 to 1.85 ksf during the test.

As shown in Plate M-1, the consolidation was allowed to proceed for four hours prior to performing the major load test. During this period, the radial total pressure decreased from 35.1 ksf to 30.4 ksf and the pore pressure decreased from 32.6 ksf to 23.5 ksf, indicating an increase in the radial effective pressure from 2.5 ksf to 6.9 ksf.

The results of the initial load test after consolidation are shown in Plate M-3. The maximum shear transfer recorded during the test was 2.14 ksf, with the value at yield being somewhat less, 2.05 ksf.

Because of the failure to obtain the maximum value of pore pressure during this experiment, it had been decided to repeat the experiment in another boring. This being the case, a complete suite of load tests were not performed. The probe was therefore loaded to failure in compression, and a series of controlled-displacement two-way cyclic tests were performed.

The results of the initial series of cyclic tests are shown in Plate M-4. The initial loading to failure in tension is included in the plate as the first half-cycle of loading. The maximum peak shear transfer during these cycles was obtained on the second failure in tension after load reversal. The value, as shown in the plate, was 2.38 ksf.

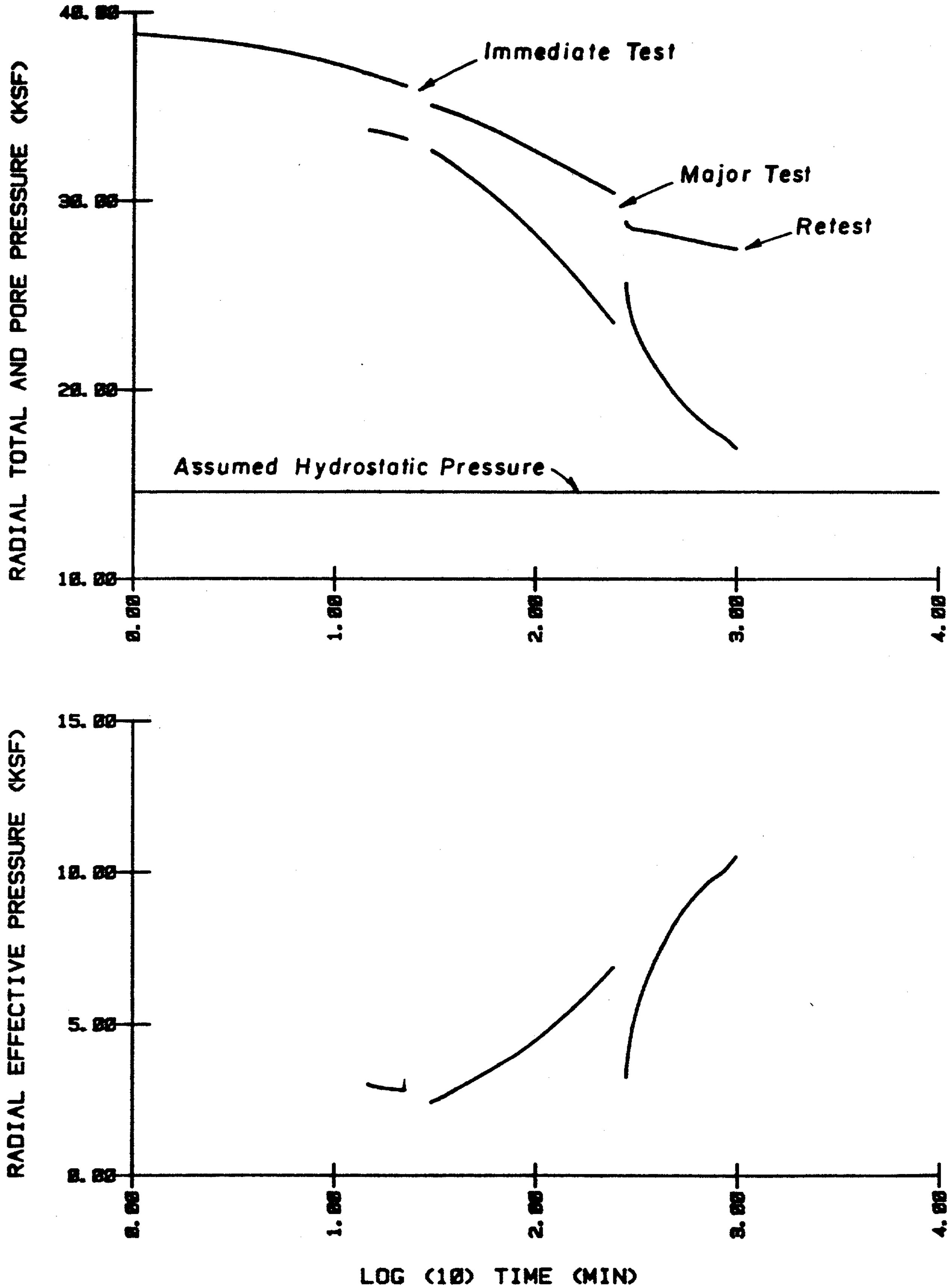
The rate of loading was then increased, with the results given in Plate M-5. The plate includes one half-cycle of loading at the slow rate, for a direct comparison. The maximum shear transfer, just prior to load reversal on the third cycle at the fast rate, was 2.03 ksf, at a rate of slip of 0.0282 in./sec.

One cycle of loading was then performed at the slow loading rate, as shown in Plate M-6. The shear transfer during the initial loading was 1.83 ksf; after the load reversal, the maximum shear transfer was 1.75 ksf, at a slip rate of 0.0144 in./sec.

The fluctuations in soil pressure during the two-way cyclic tests are shown in Plate M-7. As seen in the plate, the fluctuations in the pressures were quite large. The maximum value of radial effective pressure, recorded during the fast-rate cyclic tests, was 10.5 ksf, considerably greater than the value of 3.3 ksf which was recorded during the quiescent period following the tests.

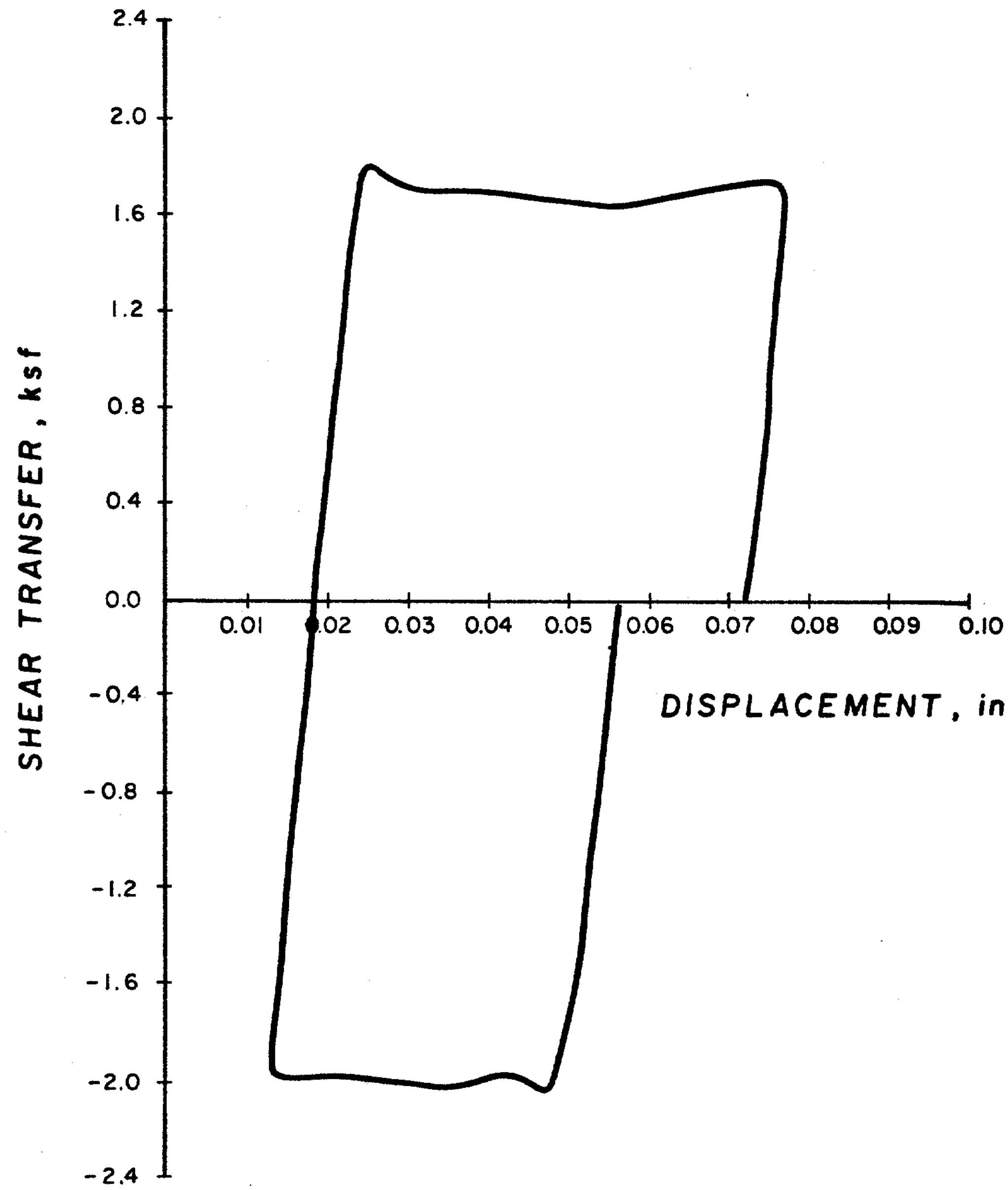
As seen in Plate M-1, the soil pressures recovered rapidly after the end of the test. During a 12-hour period of reconsolidation, the total radial pressure decreased from 28.9 ksf to 27.4 ksf, the pore pressure decreased from 25.6 ksf, indicating an increase in the radial effective pressure from 3.3 ksf to 10.5 ksf.

After the 12-hour rest period, the probe was loaded to failure, and then removed from the boring. The peak shear transfer at first yield was 1.99 ksf; the maximum value after the load reversal was 2.37 ksf, just prior to the final load reversal.

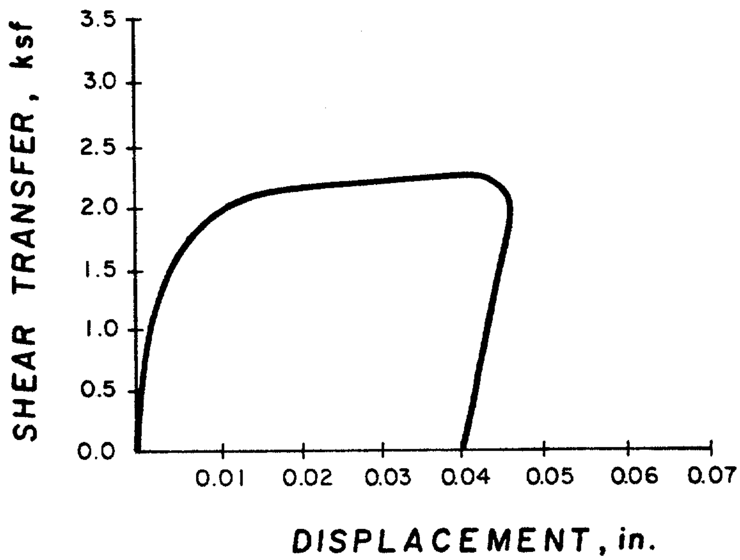


VARIATIONS OF SOIL PRESSURES DURING CONSOLIDATION

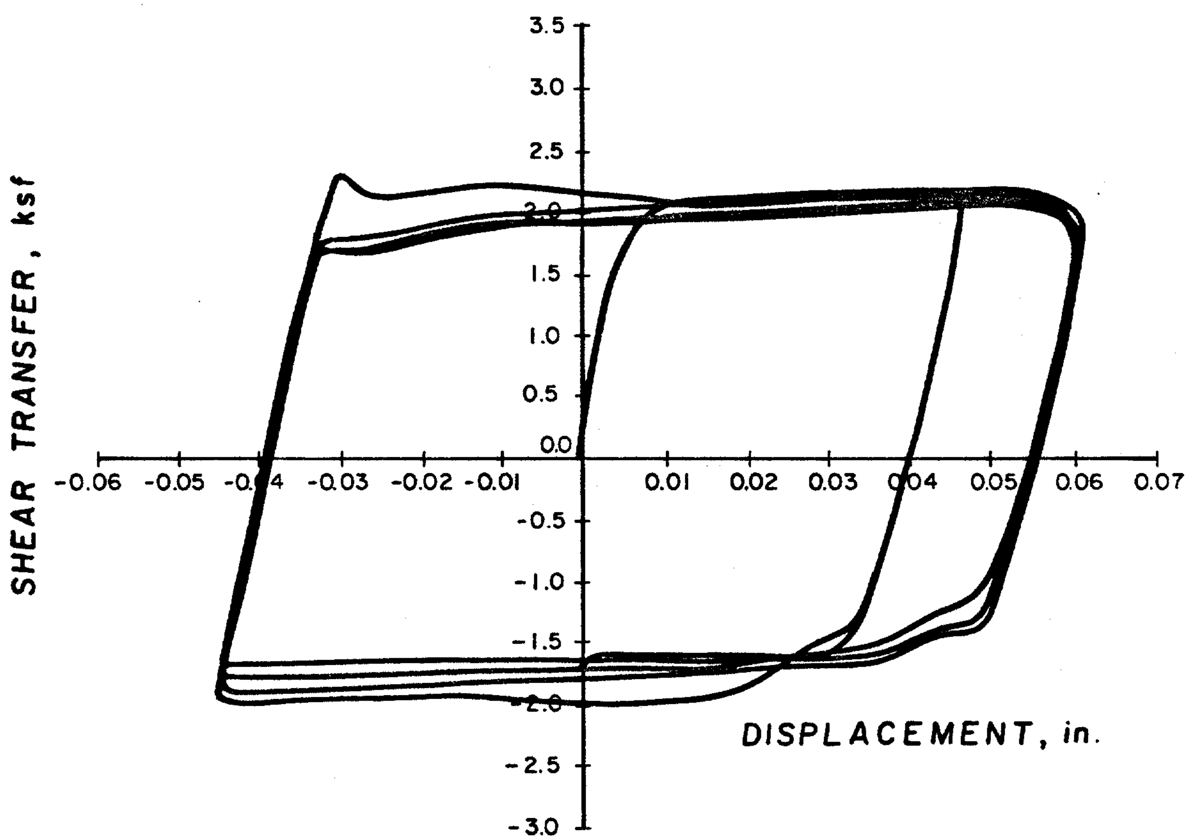
The Earth Technology Corporation



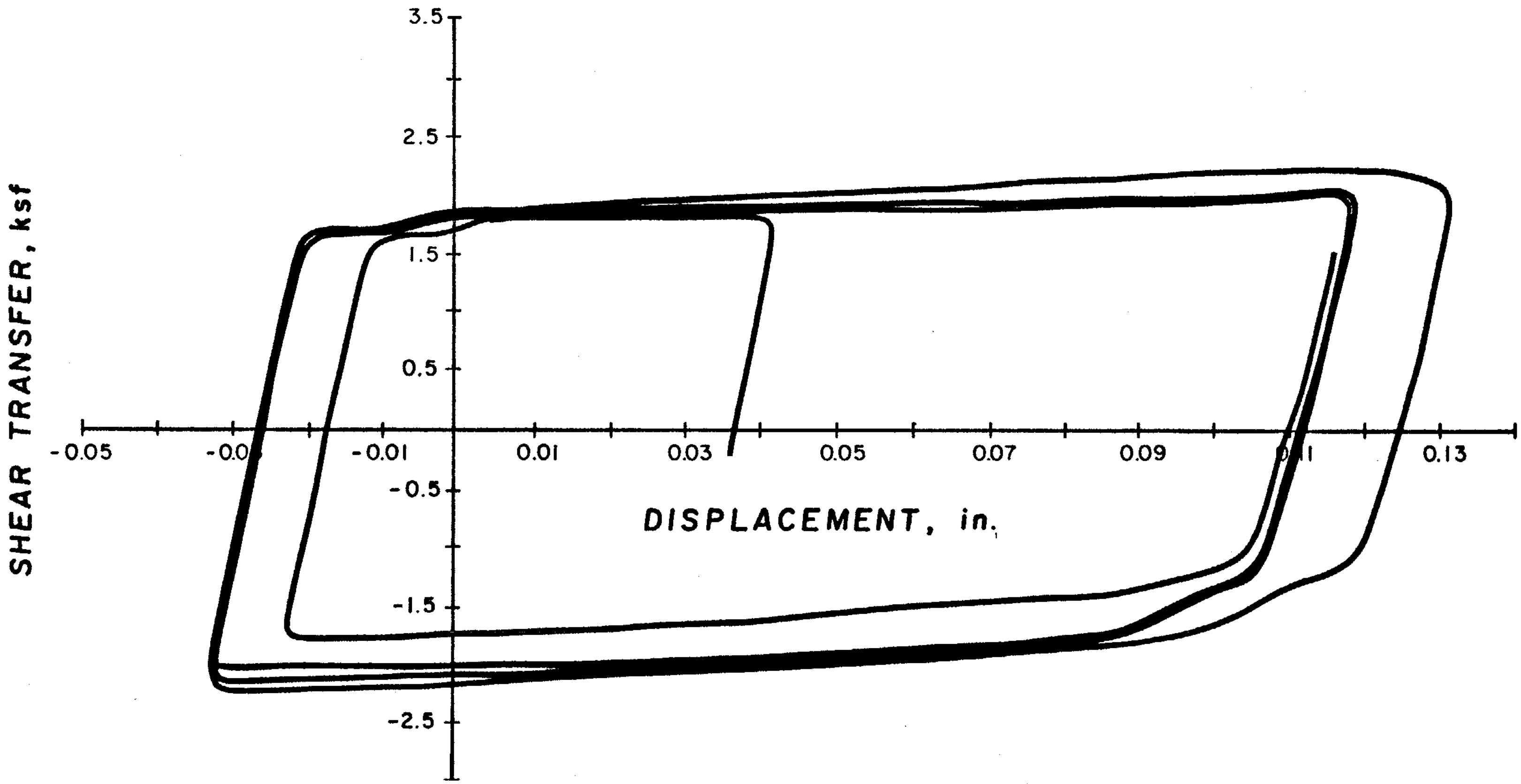
RESULTS OF THE IMMEDIATE TENSION TEST TO FAILURE



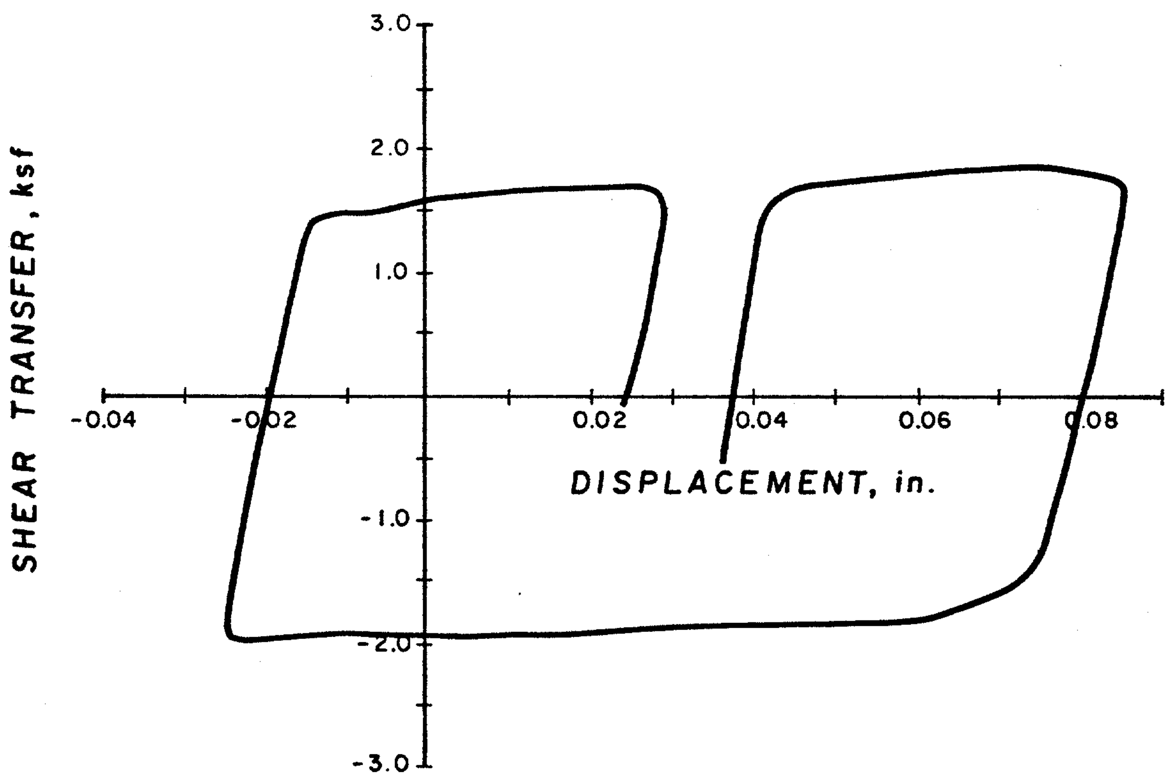
RESULTS OF THE INITIAL LOAD TEST AFTER CONSOLIDATION



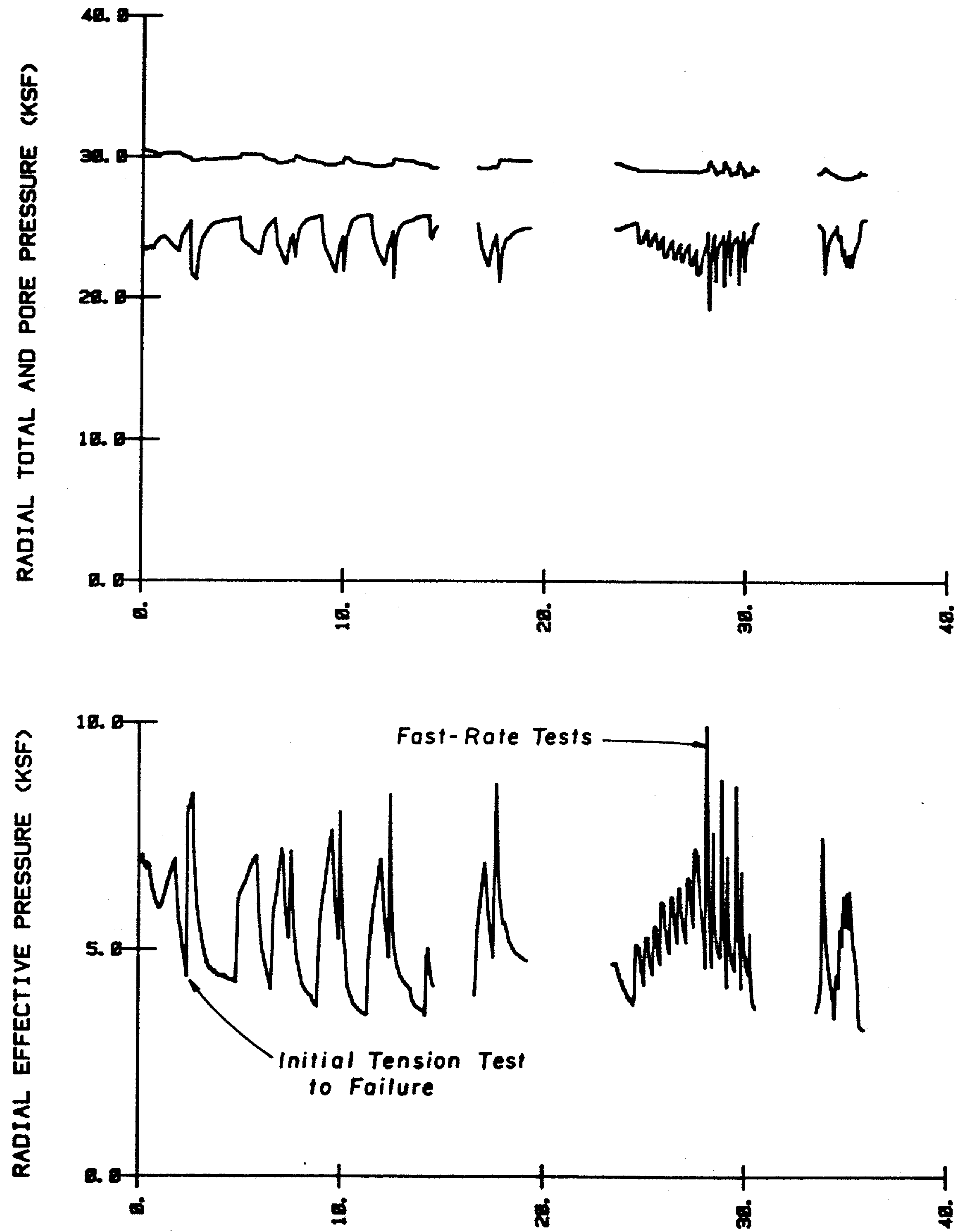
RESULTS OF THE INITIAL TWO-WAY CYCLIC TEST



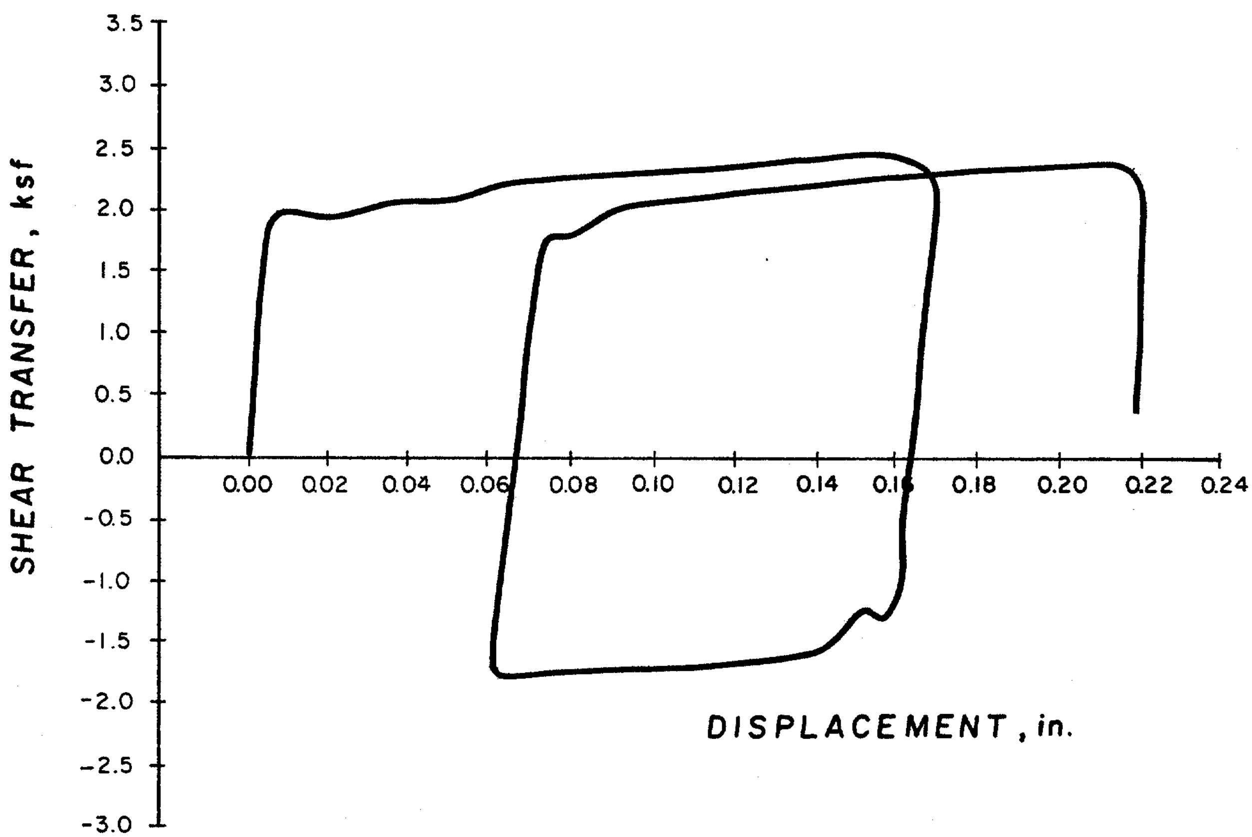
RESULTS OF THE FAST RATE TWO-WAY CYCLIC TEST



RESULTS OF THE REPEAT TWO-WAY CYCLIC TEST



FLUCTUATIONS IN SOIL PRESSURES DURING THE TWO-WAY CYCLIC TESTS



RESULTS OF THE TEST AFTER ADDITIONAL CONSOLIDATION

APPENDIX N: X-PROBE EXPERIMENT AT THE 230-FT DEPTH

RESULTS OF THE X-PROBE EXPERIMENT AT THE 230-FT DEPTH

The experiment with an X-probe at the 230-ft depth was performed during the period from 1600 hours on 24 April until 0800 hours on 25 April, 1984.

After the experiment with the first X-probe at the 229.5-ft depth, the second X-probe, which had been kept as a spare, was used at the two lowest depths. The amplifier gain on the pore pressure circuit was reduced, so that the higher excess pore pressures at these depths could be recorded. The amplifier gain in the first X-probe was not changed, so that the probe calibration could be verified upon ourreturn to the Houston office.

The probe was installed by the drilling rig, and, during the push, several different layers of soil were encountered. For some of the layers, the back wheels of the drilling rig were picked up off the ground before the probe was pushed through the layer. The layer of dense soil that had been noted in the first boring at this depth was not encountered during the push; however, the 6-in. extension on the tip of the probe had been removed.

The variations in the soil pressures during consolidation are shown in Plate N-1. The maximum radial total pressure recorded after installation of the probe was 40.6 ksf; the maximum pore pressure was 33.7 ksf. As seen in the plate, the total and pore pressures both decreased during consolidation, being 37.9 ksf and 29.5 ksf, respectively, at the time of the first load test.

The first load test was performed 18 minutes after installation, with the results given in Plate N-2. As shown in the plate, the shear-displacement response was similar to that observed in the soil at the 210-ft depth. The maximum shear transfer, on the first failure, was 0.92 ksf. During the second failure in tension, the shear transfer increased from 0.61 ksf at the first yield to 0.84 ksf, just prior to load reversal.

Consolidation was then allowed to proceed for four hours prior to performing the series of load tests. As seen in Plate N-1, the total pressure decreased from

36.8 ksf to 30.9 ksf during this period, and the pore pressure decreased from 30.0 ksf to 18.5 ksf. The radial effective pressure thus increased from 6.8 ksf to 12.4 ksf, just prior to the load test.

The results of the initial loading to failure in tension are given in Plate N-3. Because of the unusual nature of the response during this loading, it was decided to immediately proceed with the two-way cyclic tests. The soil was obviously not a clay, and therefore, the one-way cyclic tension tests were felt to be of limited benefit. During the reversal of loading, it was found that failure could not be attained in compression.

Because the X-probe could not be pushed to the intended penetration depth during the previous experiment, the 6-in. long anchoring foot extension was not used. However, this led to the inability of keeping the LVDT foot secure, resulting in the apparent downward displacements at low values of shear.

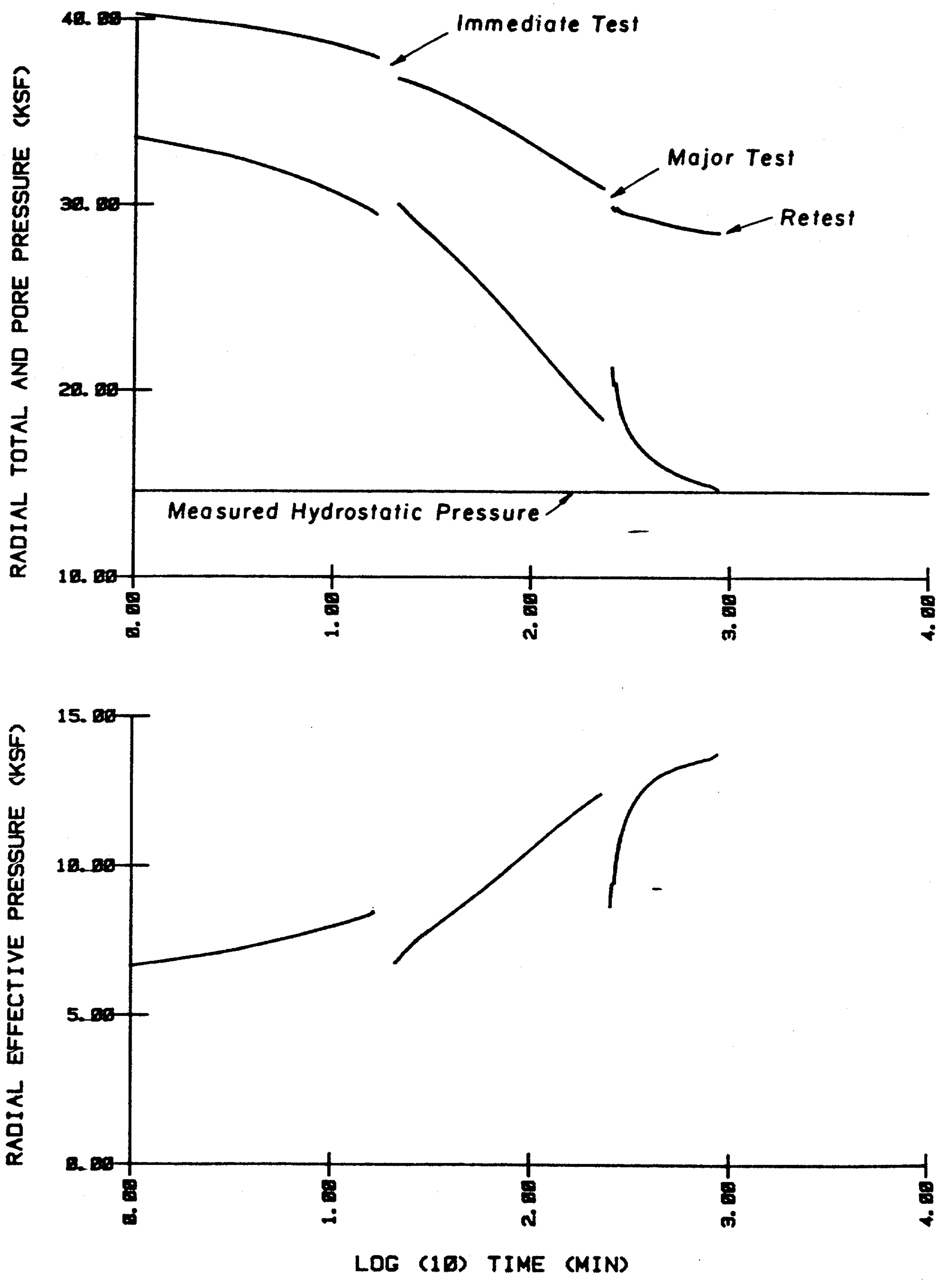
In its present form, the X-probe is not instrumented to measure end-bearing. The load applied to the N-rod string during the attempted compression tests can be estimated from the end area of the piston and the hydraulic pressure; the force was approximately 35 kips. Under such a force, the N-rod string buckles against the casing, preventing the force from being transmitted to the probe. Only two attempts were made to fail the soil in compression; it was obvious that more attempts would be futile. The probe was thus allowed to rest for a period of 11 1/2 hours, and was again loaded to failure in tension.

The peak values of shear transfer during the initial loading in Plate N-3 are 2.35, 2.37, and 2.38 ksf. The stick-slip nature of the shear-displacement behavior is, again, a consequence of the load being applied at the end of a "long elastic spring" (the N-rods).

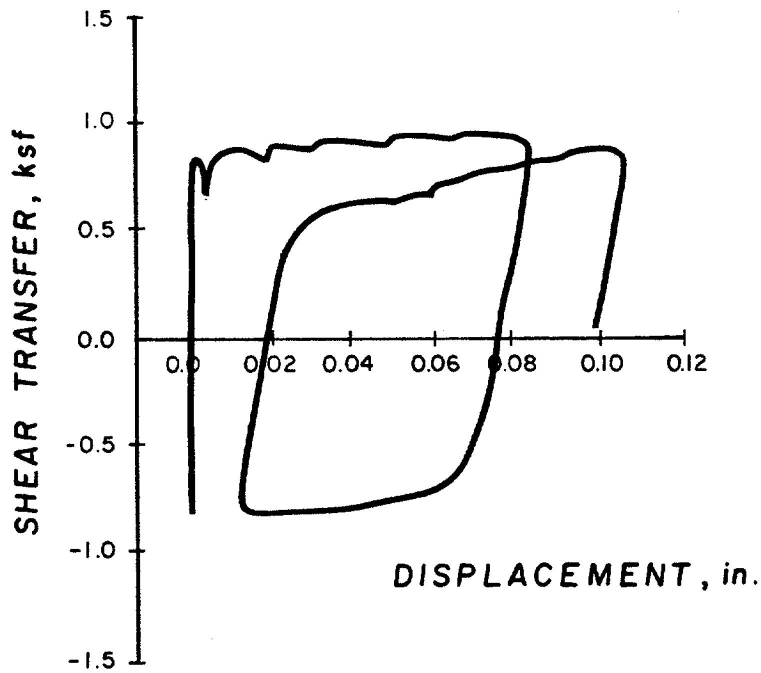
During the additional period of consolidation, the total radial pressure decreased from 29.9 ksf to 28.5 ksf. The pore pressure also decreased, from 21.2 ksf to 14.6 ksf. The increase in the radial effective pressure during this period was, therefore, from 8.7 ksf to 13.9 ksf.

The results of the load test to failure in tension are shown in Plate N-4. The shear transfer at first yield was 2.88 ksf, at a displacement rate of

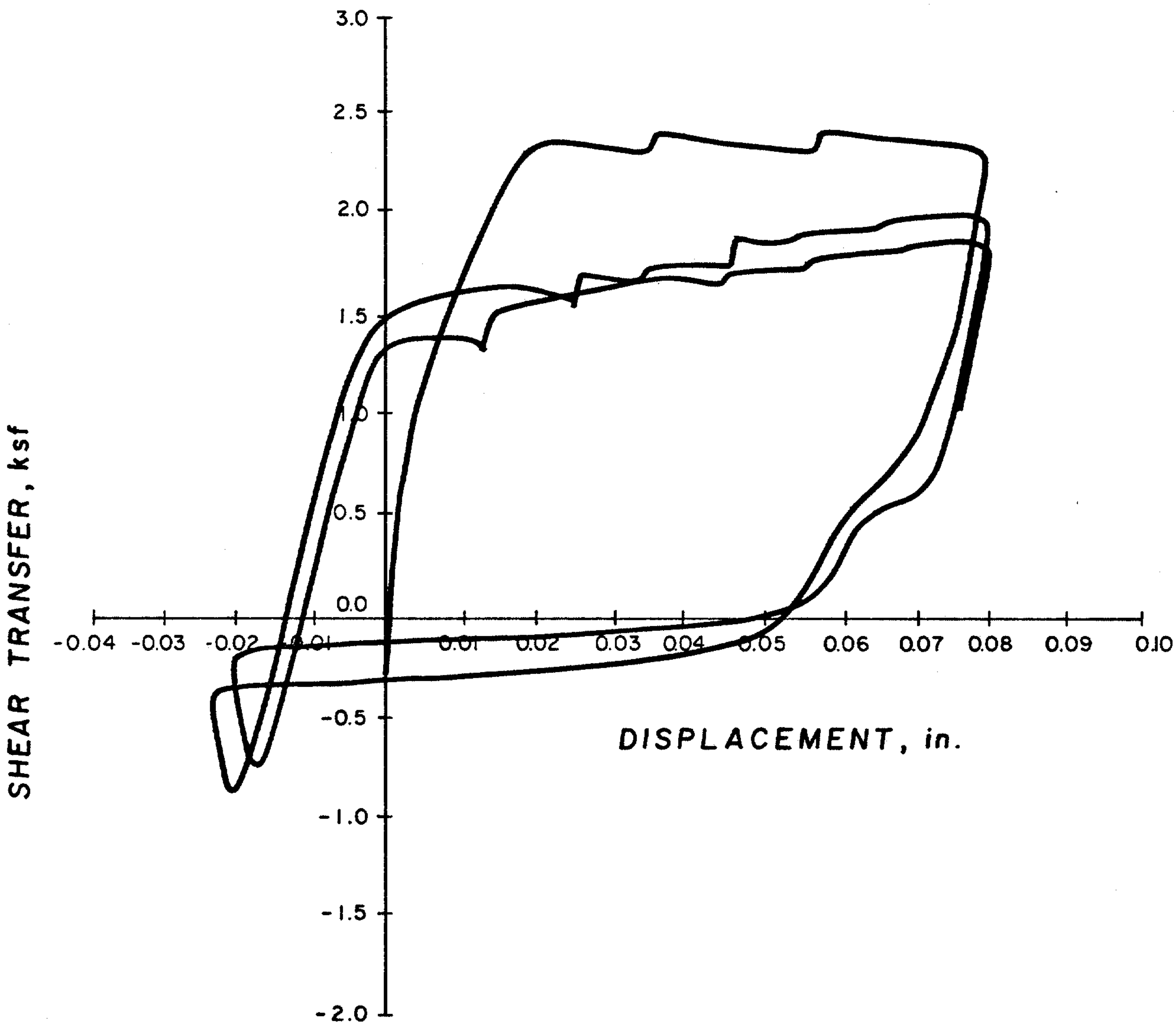
0.0022 in./sec. The rate of slip was then increased to 0.0146 in./sec, with a resulting increase in shear transfer to 3.45 ksf. This indicates a rate effect of 24 percent per log cycle of increase in the rate of slip.



VARIATIONS OF SOIL PRESSURES DURING CONSOLIDATION

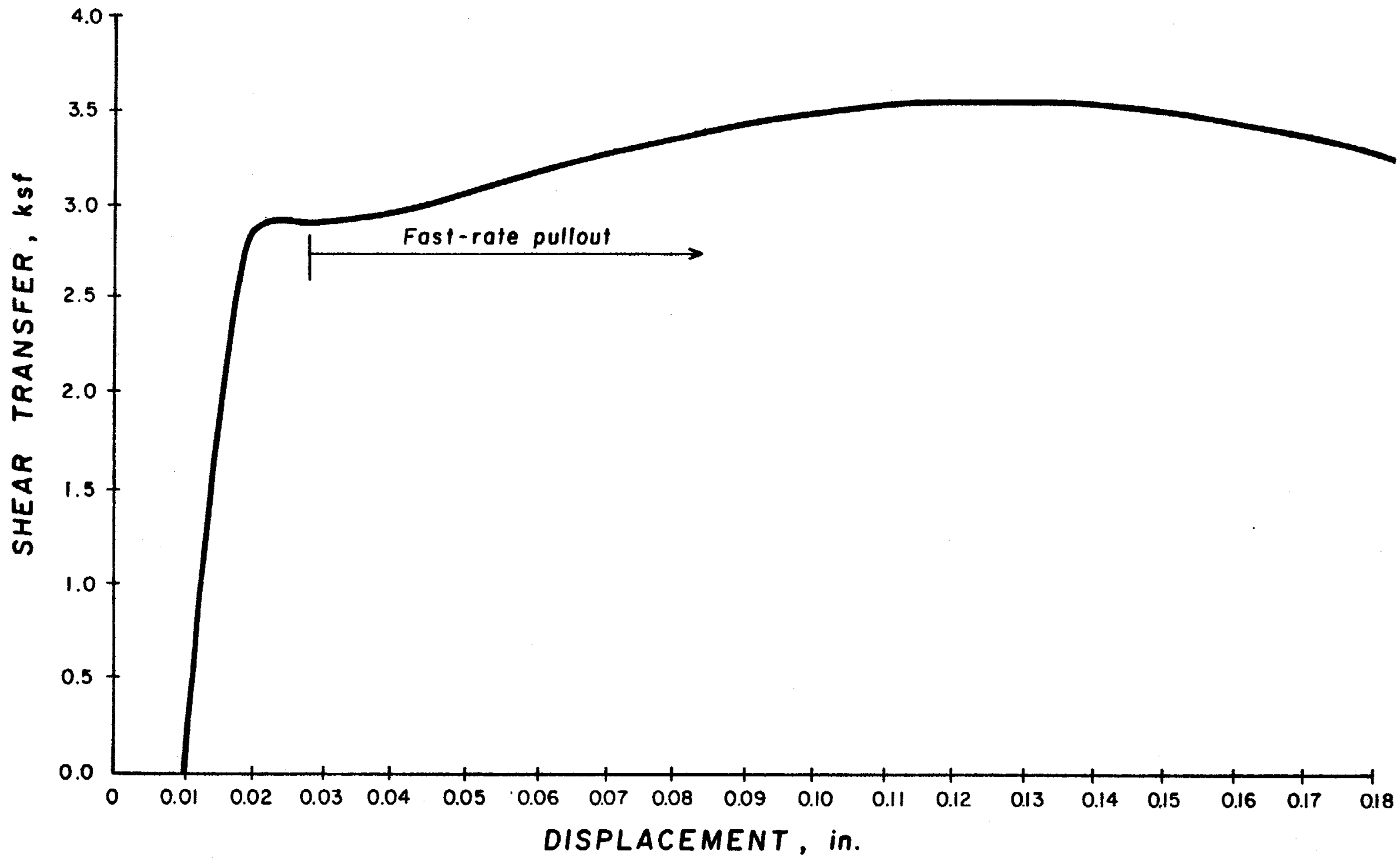


RESULTS OF THE IMMEDIATE LOAD TEST



RESULTS OF THE INITIAL LOAD TEST AFTER CONSOLIDATION

JOB NO.________



RESULTS OF THE PULL-OUT TEST AFTER ADDITIONAL CONSOLIDATION

The Earth Technology Corporation

APPENDIX O: X-PROBE EXPERIMENT AT THE 250-FT DEPTH

RESULTS OF THE X-PROBE EXPERIMENT AT THE 250-FT DEPTH

The experiment with an X-probe at the 250-ft depth was performed during the period from 1100 hours on 25 April until 1530 hours on 26 April, 1984.

The probe was installed by pushing, using the drilling rig. The time of installation was 11:19.

The variation in soil pressures during consolidation are shown in Plate O-1. The maximum radial total pressure after installation of the probe was 45.0 ksf. The maximum pore pressure was 37.8 ksf. At the time the first load test was performed, 22 minutes after insertion, the total pressure had decreased to 43.1 ksf; the pore pressure had decreased to 35.2 ksf. The corresponding increase in the radial effective pressure was from 7.2 ksf to 7.9 ksf.

The results of the first load test are shown in Plate O-2. During this experiment, the 6-in. extension for the X-probe tip was not used: the consequence is that the displacements measured during this test are not meaningful. The peak shear transfer recorded during the test (seen as the spike at zero displacement) was 0.66 ksf; the residual shear transfer after the reversal of load was 0.50 ksf.

As shown in Plate O-1, consolidation was allowed to proceed for 26 hours prior to performing the major series of load tests. During this period, the radial total pressure decreased from 43.2 ksf to 36.1 ksf and the pore pressure decreased from 35.8 ksf to 21.1 ksf. The corresponding increase in the radial effective pressure was from 7.4 ksf to 15.0 ksf.

The results of the initial loading to failure in tension are given in Plate O-3. The peak shear transfer on this loading was 1.77 ksf; the residual value of shear transfer was 1.72 ksf.

A period of 33 minutes was then allowed for the pore pressures to recover. At the end of the load test, the total pressure was 35.5 ksf; at the end of the 33-minute period, the total pressure was still 35.5 ksf. The pore pressure during this period decreased from 23.3 ksf to 22.6 ksf.

Since the time was now 14:00, and the electrical power was to be turned off at 16:00, it was decided to proceed with the one-way cyclic tension tests, so that the program could be completed.

The results of the cyclic tension tests are summarized in Plate O-4. As seen in the plate, problems were encountered with one of the flow-control valves, making test control difficult. Without better control of the load levels, the need for further testing was felt to be questionable. It was therefore decided to proceed with the two-way cyclic tests.

The results of the controlled-displacement two-way cyclic tests are given in Plate O-5. The large excursion on the first cycle was done to reposition the LVDT in the mid-range of travel. Since the 6-in. extension was not on the probe, the slip-joint had closed during consolidation. The peak shear on the first cycle was 1.73 ksf, near the value of 1.77 ksf measured during the initial load test.

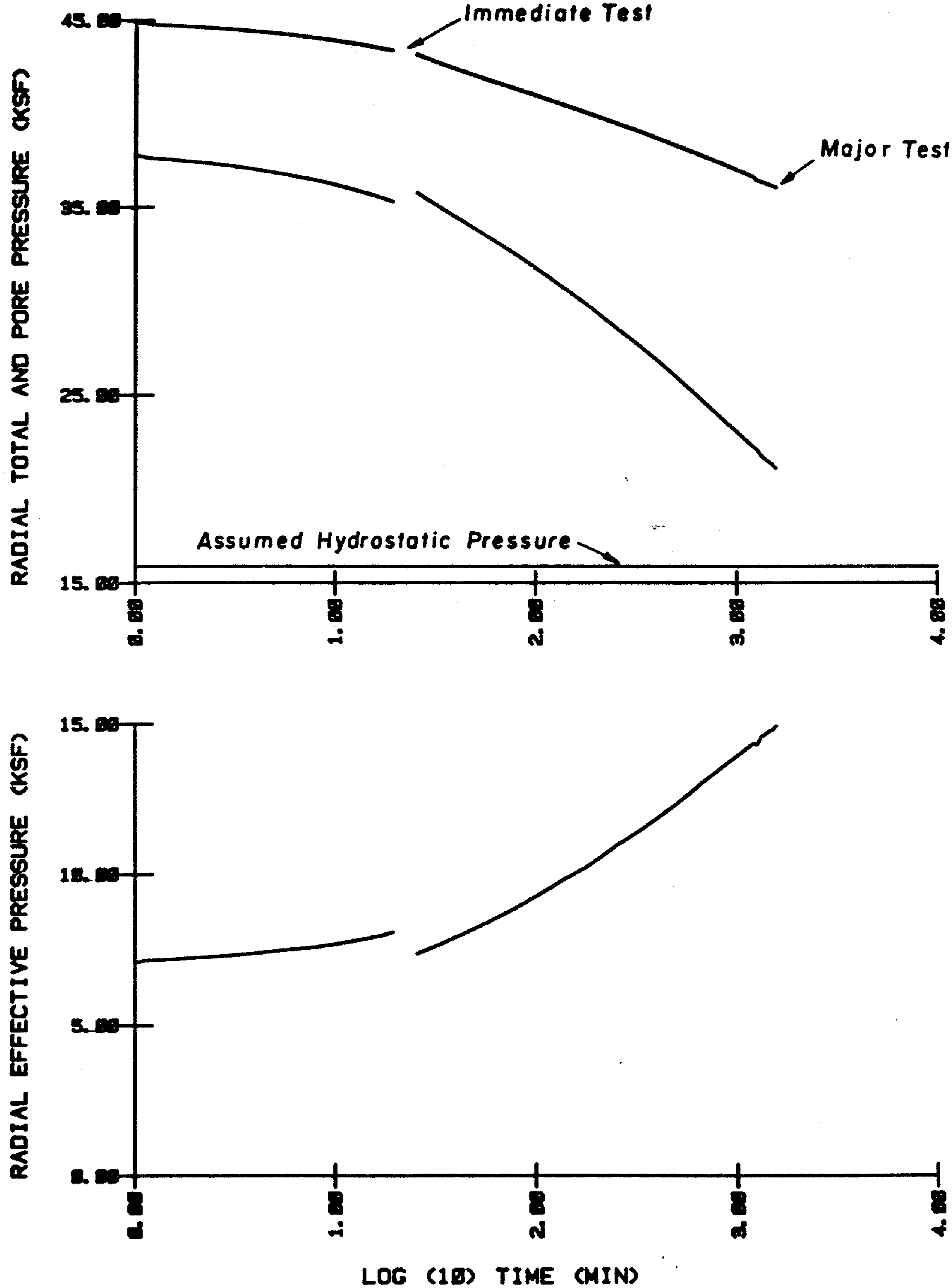
After three cycles, the shear transfer had estabilized, with a peak value of 1.51 ksf, and a residual value of 1.28 ksf. The load rate was then increased, with the first loading at the fast rate shown in the Plate O-5. The value of shear transfer during the fast-rate loading was 1.76 ksf.

The next three cycles of fast-rate two-way cyclic loading are shown in Plate O-6 The peak value of shear transfer on the third cycle is 1.56 ksf, with a residual value of 1.52 ksf, at a rate of slip of 0.0562 in./sec.

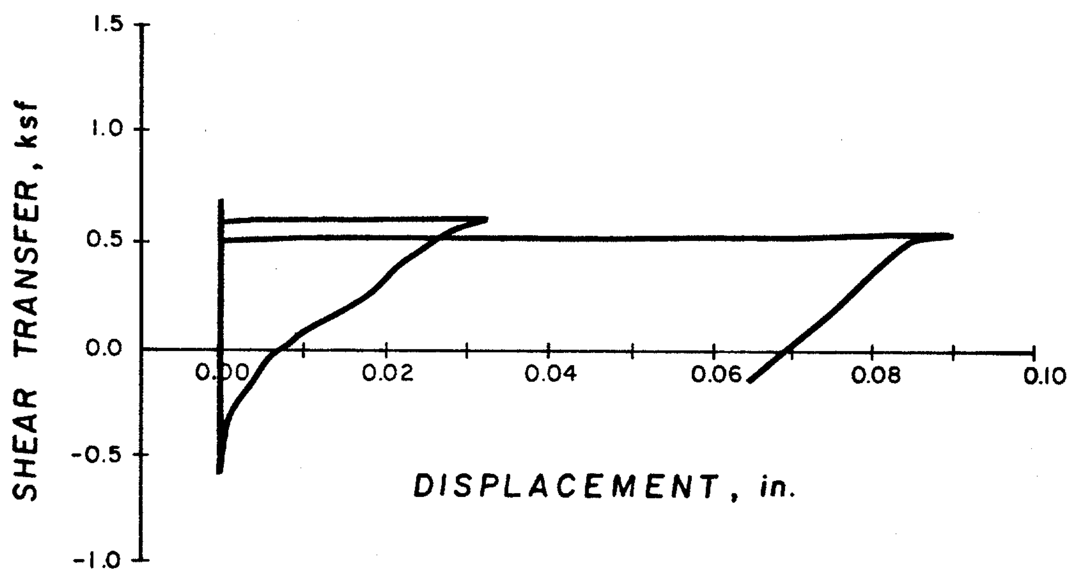
The load rate was then decreased, with the results shown in Plate O-7. The peak shear transfer in Plate O-7 is 1.37 ksf, with a residual value of 1.14 ksf. The rate of slip was 0.000791 in./sec.

During this experiment, the rate effect was thus indicated to be 19 percent per log cycle of increase in the rate of slip.

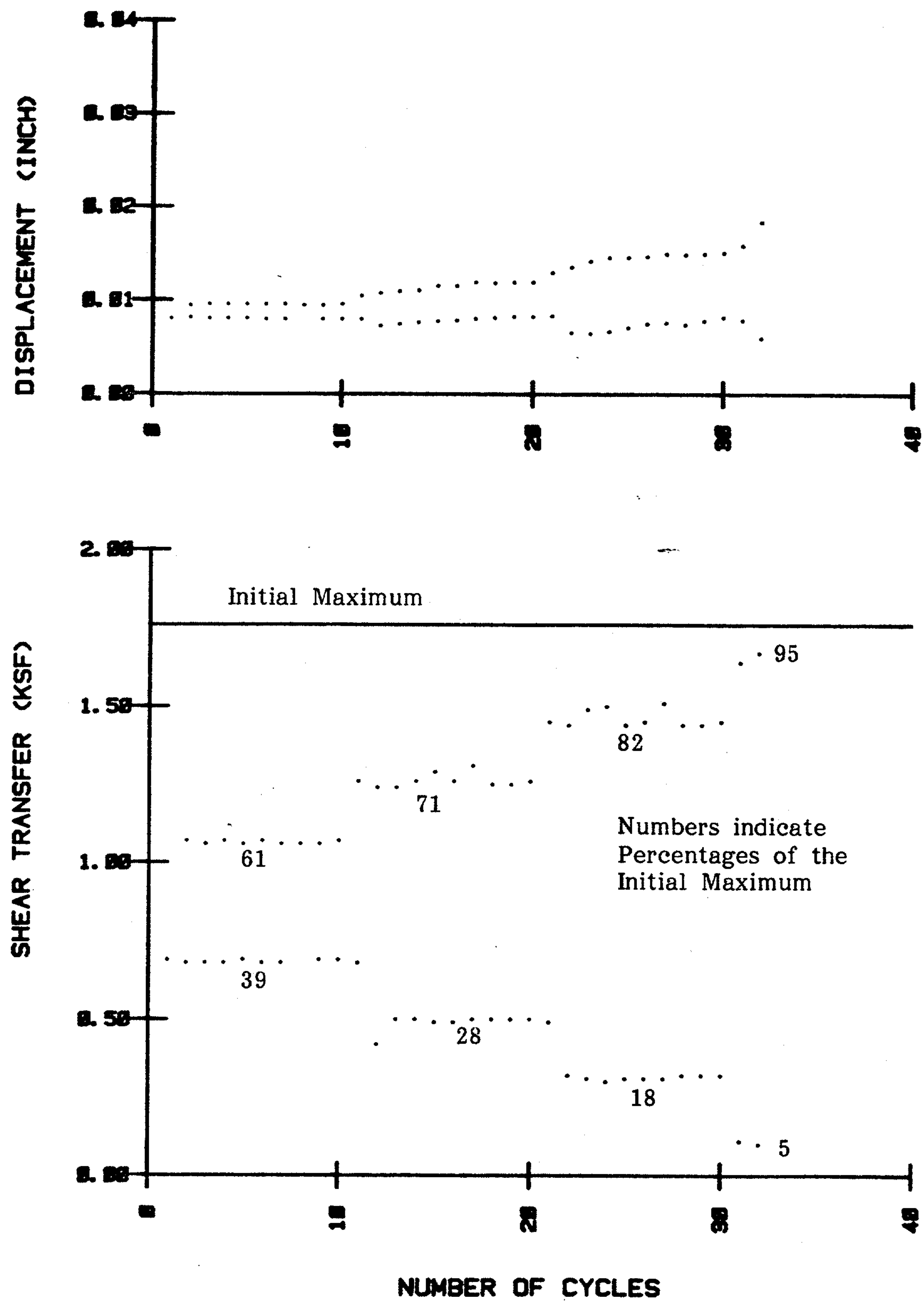
The fluctuations in soil pressure during the two-way cyclic tests are shown in Plate O-8. The patterns of behavior are the same as shown earlier; the increase in effective pressure and the decrease in pore pressure occur during the periods of constant-shear plastic slip.



VARIATIONS OF SOIL PRESSURES DURING CONSOLIDATION

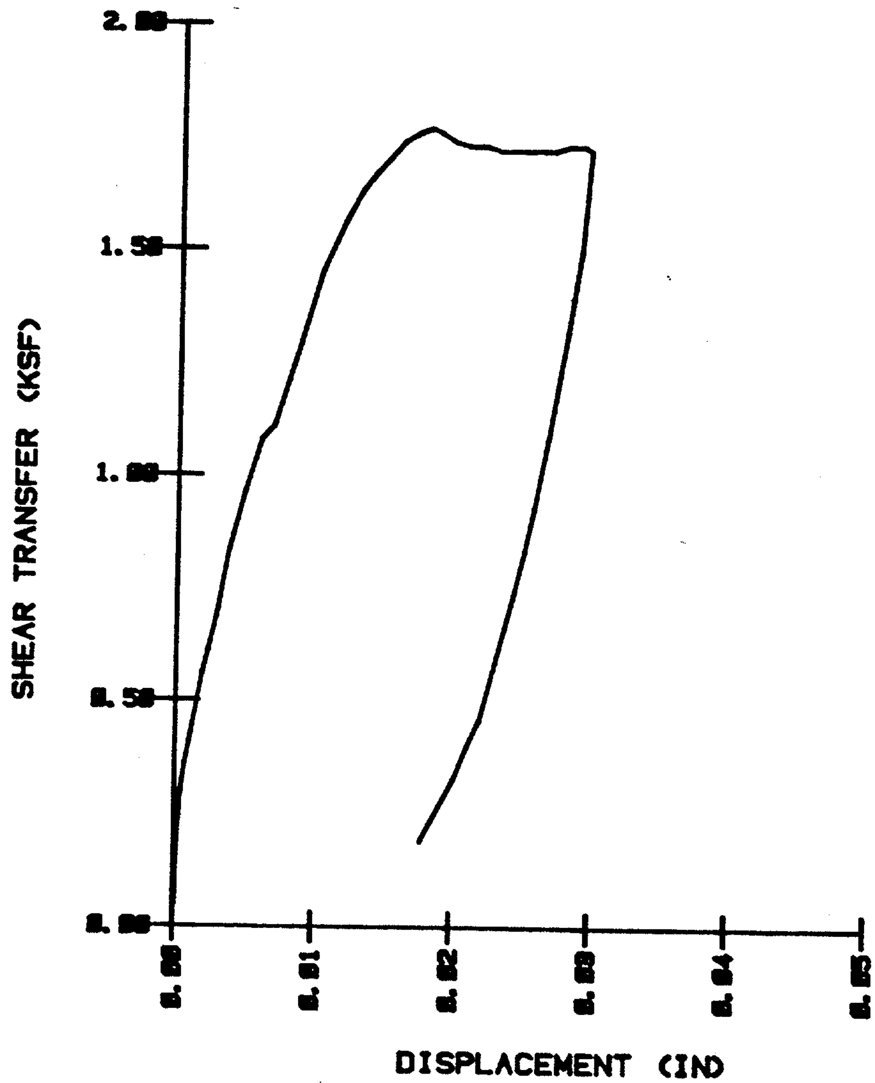


RESULTS OF THE IMMEDIATE TEST



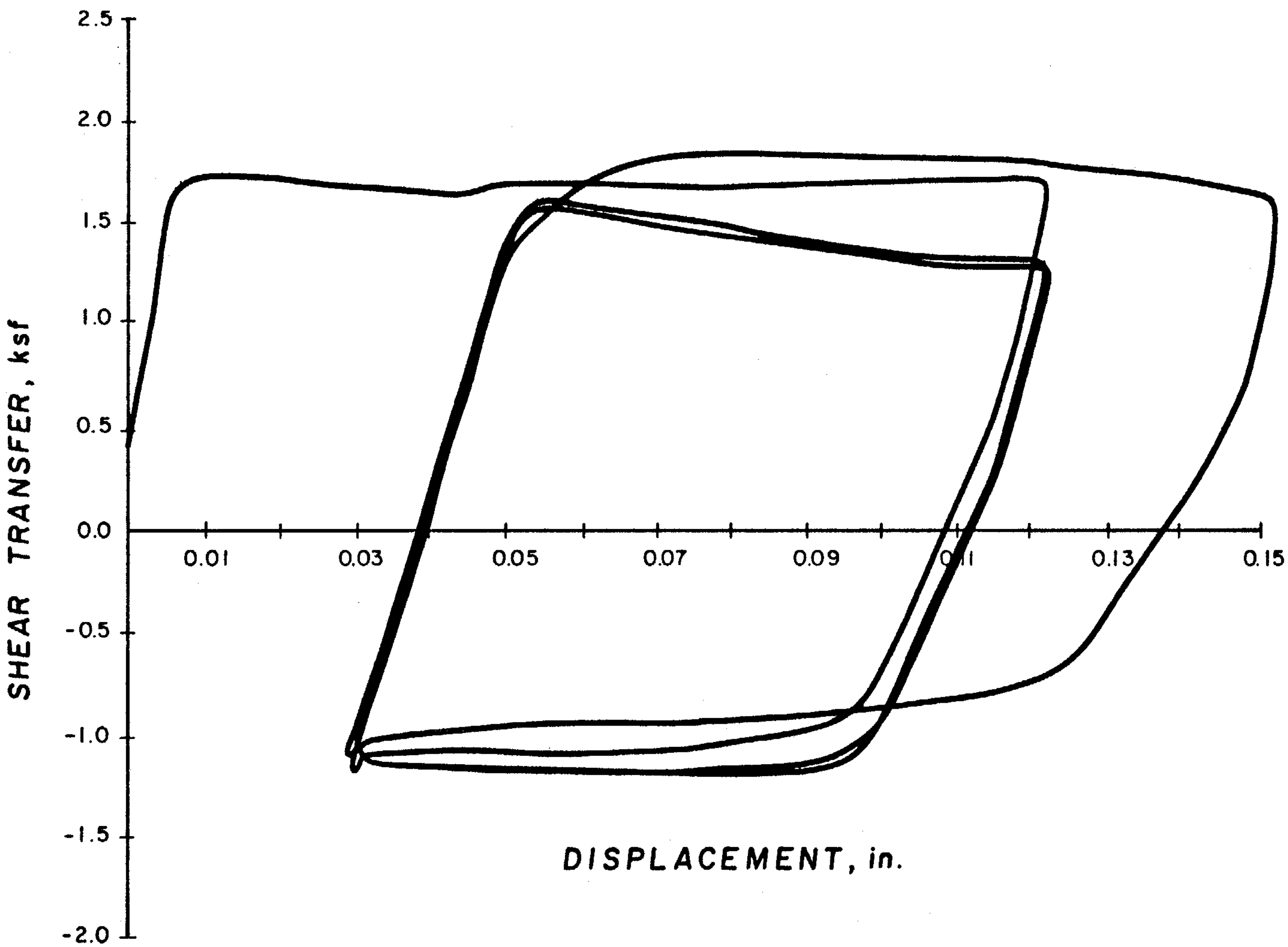
SUMMARY OF THE ONE-WAY CYCLIC TENSION TESTS

JOB NO.



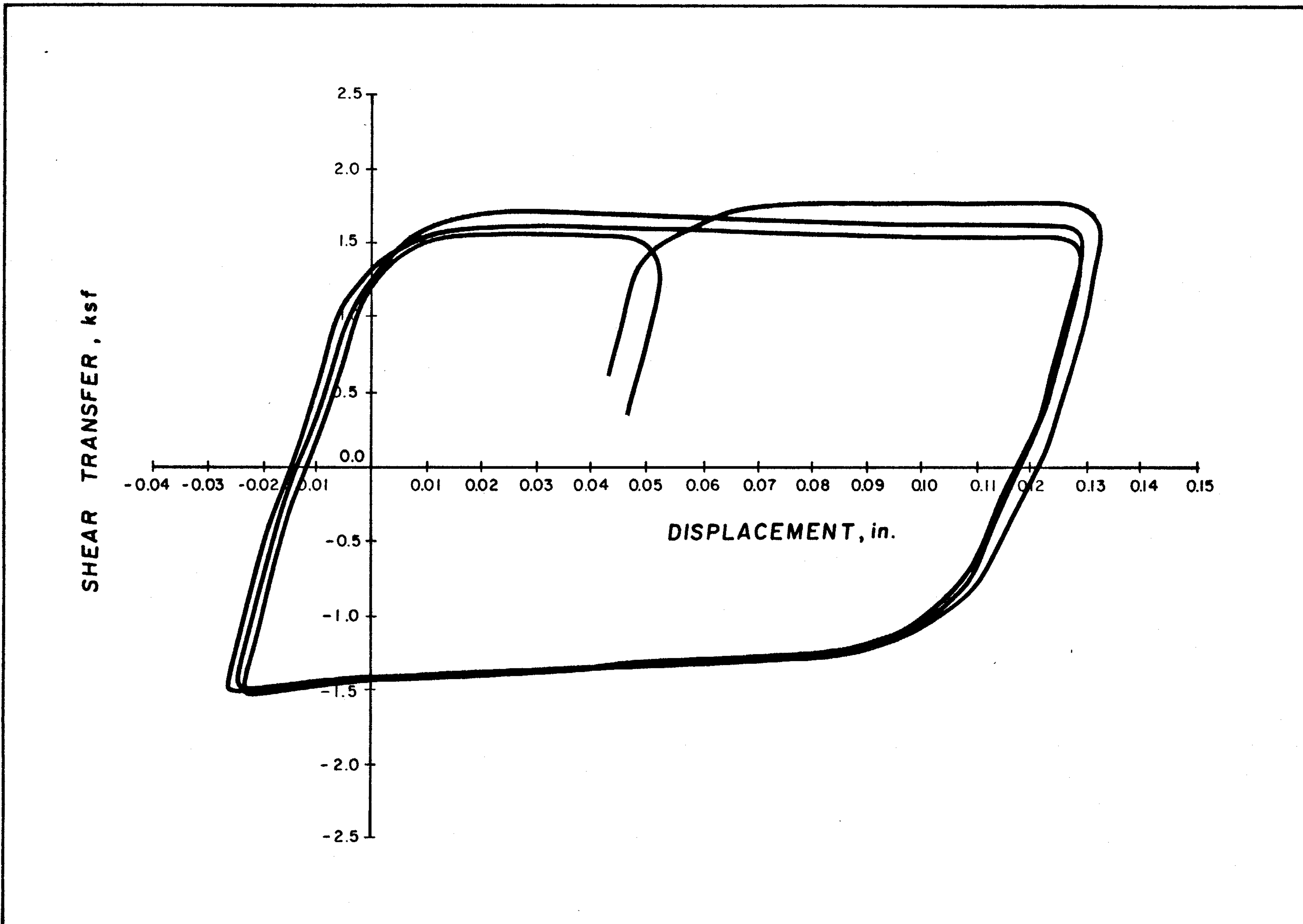
RESULTS OF THE INITIAL LOADING TO FAILURE

JOB NO.

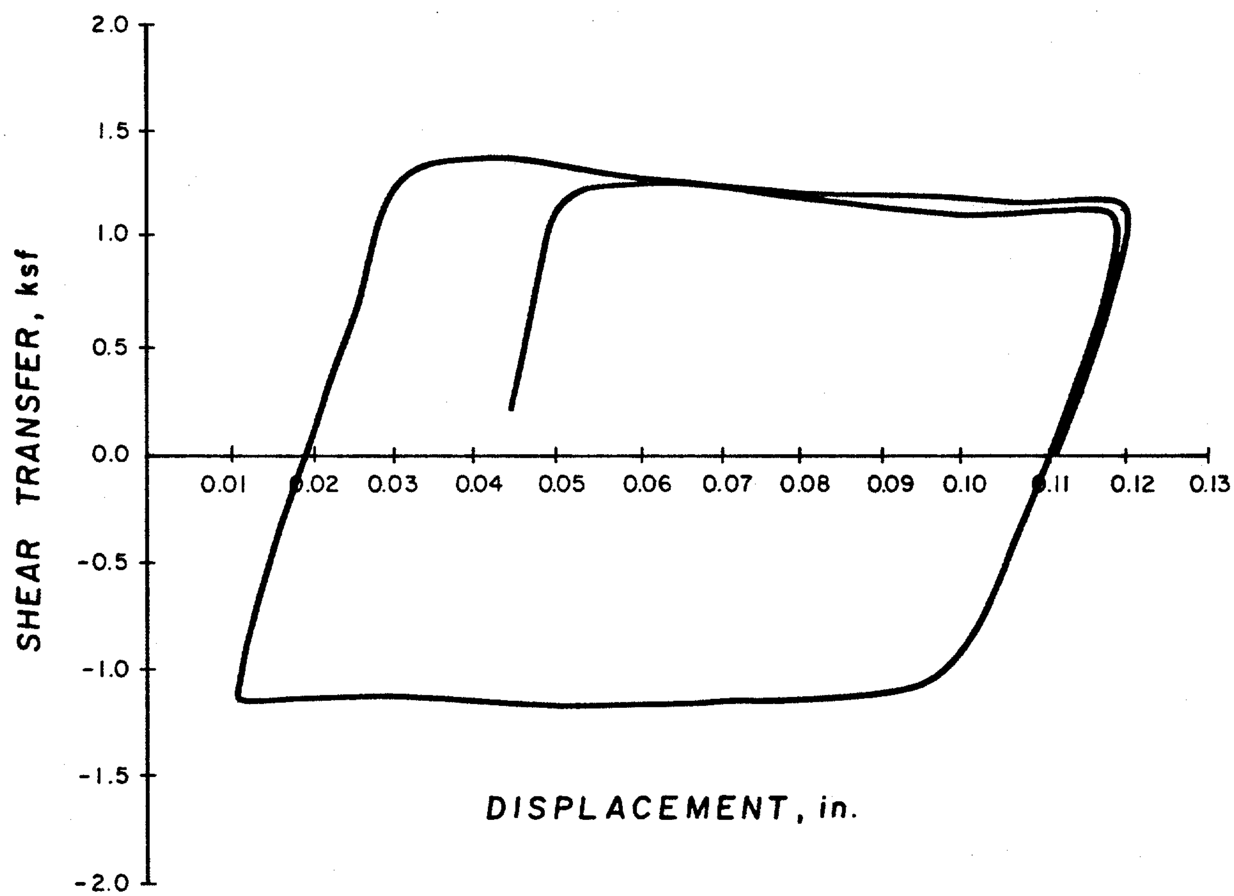


RESULTS OF THE INITIAL TWO-WAY CYCLIC LOADING TESTS

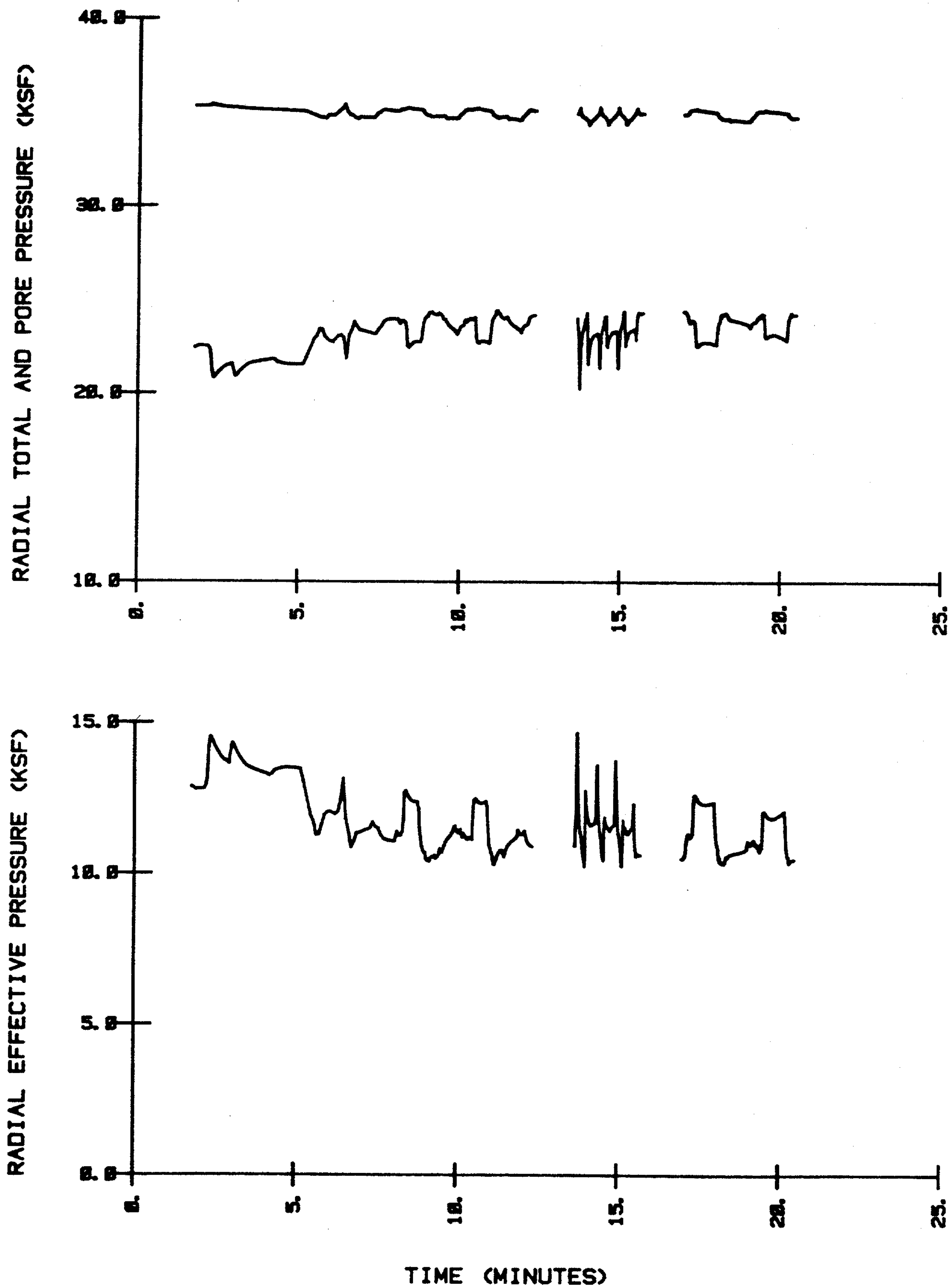
JOB NO. ________



RESULTS OF THE FAST RATE TWO-WAY CYCLIC TEST



RESULTS OF THE TWO-WAY CYCLIC TEST AFTER ADDITIONAL CONSOLIDATION



FLUCTUATIONS IN SOIL PRESSURES DURING TWO-WAY CYCLIC LOADING

The Earth Technology Corporation